

LEWIS

A Comparison of Built up
and Solid Wooden Beams

Architectural Engineering

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A COMPARISON OF
BUILTUP AND SOLID
WOODEN BEAMS.

BY

BURT A. LEWIS.

THESIS FOR THE DEGREE OF BACHELOR OF SCIENCE
IN ARCHITECTURAL ENGINEERING.

COLLEGE OF ENGINEERING.
UNIVERSITY OF ILLINOIS.

PRESENTED JUNE 1906.

A COMPARISON OF
BUILDING AND
WOODEN BEAMS

BY

JOHN A. KIRBY
CHIEF ENGINEER
U. S. NAVY

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

Burt A. Lewis

ENTITLED A Comparison of Built-up and Solid Wooden Beams

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE

OF Bachelor of Science in Architectural Engineering

N. Clifford Pickew.

HEAD OF DEPARTMENT OF Architecture

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This thesis results from a determination of the strength and probable cost of several types of built-up wooden beams and a comparison of them with solid beams. The lumber was obtained from an Urbana lumber company, supplied as short leaf pine from the Ozark mountain region in Missouri. It was the best grade of dimension stuff in this market, and, excepting a few pieces, it was remarkably free from knots shakes and sap. The amount of moisture varied from 13.7% for the most seasoned pieces to 35.8% for some very green timbers.

Three specimens of this material were tested in tension with the following results:

Cross Section.	Area.	Breaking Load.	Tension.
$1\frac{1}{4}" \times 5\frac{3}{4}"$	7.2 ^{sq} "	47500 [#]	6600 ^{#/sq"}
$1\frac{3}{4}" \times 5\frac{3}{4}"$	10.1 ^{sq} "	48900 [#]	4860 ^{#/sq"}
$1\frac{1}{4}" \times 5\frac{3}{4}"$	7.2 ^{sq} "	39700 [#]	5510 ^{#/sq"}

Four samples were weighed with the results shown in the following table:

Beam No.	Size of Sample	% Moisture	Weight	Wt. 0% Moisture	Wt. 15% Moisture
#1.	2.38 cu. ft.	35.8	88.25 [#]	27.3 ^{#/cu. ft.}	31.4 ^{#/cu. ft.}
#2.	1.41 " "	31.9	58.30 [#]	31.3 " "	36.0 " "
#3.	1.35 " "	17.8	50.00 [#]	31.4 " "	36.1 " "
#4.	1.70 " "	17.9	55.00 [#]	27.4 " "	31.5 " "



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No attempt was made to select the lumber more than would be done by any careful carpenter, such as favoring a bad stick by placing it so that the knotty portion would be in compression.

In making the tests the beams were loaded at the third points by steel rollers resting on iron plates that distributed the pressure over a part of the beam. They were supported by cast iron plates on rocker bearings, the span being 15'-6" C. to C. of supports. Deflectometers were placed under the beams at the center and at the third points, readings being taken as nearly as possible at each change of 1000[#] in the loading.

No attention was paid to the weight of beam or bearings.

Since many authorities assume $\frac{1}{360}$ of the span as the limit of deflection allowable where ceilings are to be plastered beneath the beams, then 0.52" for a span of 186" was taken for the limit of deflection for these beams,

and the load at that point was computed. The modulus of elasticity was calculated by the formula $E = \frac{23 Pl^3}{1296 \Delta I}$, deduced from the elastic curve at the center of a beam equally loaded at the third points, i.e. the two points dividing its span into three equal parts.

E = modulus of elasticity.

P = load in pounds when $\Delta = \frac{52}{18600}$ of the span.

l = span or distance c. to c. of supports, in inches.

I = moment of inertia of cross section of beam.

Δ = limit of deflection = $\frac{52}{18600}$ of the span.

The modulus of rupture was computed by the formula $R = \frac{Wl e}{6 I}$, for beams equally loaded at the two third points.

R = modulus of rupture.

W = breaking load in pounds.

e = distance in inches from neutral axis to most distant fibre of cross-section.

The longitudinal shear is computed by the formula $H = \frac{3W}{4bd}$.

H = longitudinal shear.

b = breadth of beam in inches.

d = depth of beam in inches.

The percentage of moisture was determined by cutting samples from the beams

at the time of testing and weighing them before and after drying. One hundred times the ratio of the difference between these weights to the weight of the dried sample is taken as the percentage of moisture contained in that sample.

The cost per beam is taken as the actual cost of the materials plus the cost of the labor which is assumed to be fifty-five cents per hour for one carpenter and his helper. The cost per thousand pounds safely carried by the beam is the ratio of the cost of the beam to the load in thousands of pounds taken at the point where the deflection at the center of the beam is 0.52 inches.

The plotted curves are those deflections obtained by observation at the center of the beam. Loads are plotted as ordinates and deflections as abscissae.

The other tables explain themselves.

The Beams.

Type A comprised solid beams ordered especially for these tests. #1 and #2 were tested when received, January 16 1906, and the moisture percentage was found to be 35.8 and 31.9 respectively. #3 and #4 were stored in a room heated by steam until April 14 1906, when they were tested; the moisture percentage then present being 17.8 and 17.9 respectively, showing a loss of one half the original moisture during 88 days.

Beam #1 contained a few knots near the neutral axis and it was very green when tested. The first crack appeared at a load of 20000# and at 21580# the beam broke near a knot in the lower edge. At the time of failure the beam had crushed at one loaded point for a distance of three inches into the beam.

Beam #2 had a few small knots near the neutral axis and was very green. The first crack appeared under a load of 21000#, and the beam failed by the lower fibres tearing

6.

apart near the center with a load of 27880#.

Beam #3 was badly season checked along neutral axis, some cracks being over a foot long and extending half way through the beam. The beam was quite knotty and it was so placed that most of the knots came on the compression side. The first cracking occurred under a load of 12000#, and the beam finally failed by compression at 26000#. The fibres doubled on themselves at a large knot near a loaded point. Fibres in tension remained intact. The season cracks along the neutral axis opened slightly and others appeared, but the beam did not fail by longitudinal shear.

Beam #4 was badly checked along the neutral axis with a few small knots also there. The first cracking was noticed at 13000# along the neutral axis and the check cracks seemed to widen. The beam failed at 27810# by the tension fibres tearing

apart near the center. The maximum load recorded was 28150[#], when a sliver four feet long was split entirely free from the lower corner of the beam. After another sliver had split partially free, the load receded and the beam finally broke with a loud report.

Type B was composed of three 2"x12" timbers spiked side by side with one 20d spike in every lineal foot of beam for each member, and several more at the end. The cost was:—

2 men $\frac{3}{4}$ hours @ 55 ¢/hour = \$ 42

3 pieces 2"x12"x16'-0" @ \$26.00/M = 2.50

50 spikes 20d @ $2\frac{1}{2}$ ¢/# = .04

Total for each beam = 2.96.

Beam #5 was made of clear straight grained lumber, one plank being split at one end along the neutral axis for a distance of about two feet. When the load reached 5000[#] the spike heads crushed into the wood slightly, probably showing that if the planks had been free they might have buckled under this load.

Longitudinal shear was evident at 14990#. Failure occurred at 17970# when two members split along the neutral axis from end to center, the other member still remaining entire.

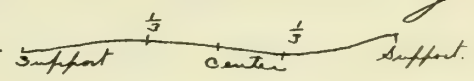
Beam #6 contained one cross-grained knotty plank but the others were good. The first loud crack occurred at about 8000#. Beam failed by the breaking of the cross-grained member near a knot at 12000#. The other members were still sound and would possibly have supported a larger load. As these two beams failed under such different loads and the plotted curves show wide divergencies, a third beam 5a was built and tested.

Beam #5a was made of clear straight grained timber. First cracking was heard at 11000#. The beam failed at 21550# by the splitting of one plank along the neutral axis, the other two being still unbroken.

Type C was composed of six 1x12's nailed together with three 10d nails per lineal foot for each board.

The cost was assumed as follows:-

2 men $\frac{1}{2}$ hours	@ 55¢/hour	= \$.42
6 pieces 1" x 12" x 16'-0"	@ \$28.00/M	= 2.68
300 nails 10d	@ 2½¢/#	= .10
Total for each beam		= 3.20.

Beam #7 was composed of good clear and straight grained lumber but it buckled under 14400# assuming the form here shown: 

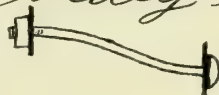
Beam #8 was clear and straight grained. One of the boards cracked near a small knot at 15500#. The beam failed by buckling at 16310# taking a form similar to that of #7.

Type D was composed of six 2" x 6"s spiked on top of each other with two 20d spikes in each member every six inches in length, and six $\frac{1}{2}$ " bolts between each load point and support. The following was assumed as cost.

2 men 1½ hours	@ 55¢/hour	= \$.69
6 pieces 2" x 6" x 16'-0"	@ \$24.00/M	= 2.31
180 spikes 20d	@ 2½¢/#	= .15
12 bolts $\frac{1}{2}$ " x 11"	@ 3½¢ each	= .41
24 washers		= .08
Total cost of one beam		= 3.64.

Deflectometers were not used but readings taken with a carpenters rule.

Beam #9. The first crack was heard at 1000#. The deflection was about 10.35 inches when the load was 8000# and none of the timbers had broken. When the load was taken off, the beam was still deflected $7\frac{1}{2}$ inches. A bolt was cut out of this beam and it was found to be badly bent in the form here shown.



Beam #10. The first crack was heard at 7000#. The deflection was 10.20 inches under a load of 8000#. When the load was removed the beam was still deflected $4\frac{3}{4}$ inches. The bolts and spikes were badly bent as in #9.

Type E was built of nine thicknesses of 1"x8" and 1"x4" nailed with six 10d nails per lineal foot for each thickness of boards. The lumber was very knotty. The cost was one and one-half times the cost of type F = \$5.13

Beam #11 buckled at 18100#, the first crack was heard at 13000#. The form assumed by the buckled beam was

similar to that of #7.

Beam #12 buckled at 18200#. The first crack was heard at 14000#, and longitudinal shear appeared at 18050#. This beam buckled in a very peculiar manner. When placed in the machine the center, load points and supports were all in a straight line. When it buckled the center had passed six inches sidewise leaving the supports and load points still in the same straight line.

$\frac{L}{2}$ $\frac{L}{2}$
 Support Center Support.

Type F was built of the same material and in the same manner as type E, except that there were only six thicknesses of boards instead of nine. The cost is about as follows:—

2 men	1 $\frac{1}{4}$ hours	@ 55¢/hour	= \$.69
3 pieces	1" x 8" x 16'-0"	@ \$25.00/M	= 2.40
400 nails	10d	@ 2 $\frac{1}{2}$ ¢/#	= .33
Total cost of one beam			= 3.42

Beam #13 buckled at 10300# taking a form similar to #7. The first crack occurred under 4000# load.

Beam #14 buckled at 10500# the first crack occurring at 7000#.

Type G was composed of three 4×4 's, having 1×6 " pieces nailed to side at an angle of 45° with the 4×4 's. Three 10d nails were driven through each 1×6 " into each 4×4 ". On opposite sides the 1×6 " pieces were placed perpendicular to each other. It was thought that three 4×4 's instead of two 4×6 's would lessen the longitudinal shear, because at the distance of the longitudinal joint from the neutral axis the shearing stress would only be $\frac{5}{6}$ of that found at the neutral axis. Below is the cost assumed per beam.

2 men	$1\frac{3}{4}$ hours	@ 55¢/hour	= \$.76
3 pieces	$4 \times 4 \times 16'-0"$	@ \$28.00/M	= 1.30
5 "	$1 \times 6 \times 16'-0"$	@ 25.00/M	= 1.00
400 nails	10d	@ $2\frac{1}{2}$ ¢/#	= .32
Total cost for each beam			= 3.38.

Beam #15, lower 4×4 " broke in tension when load was 7900#, the other two 4×4 's were still sound. The 1×6 " pieces split badly at the nails.

Beam #16. The lower member broke at load of 7410#, same as in #15.

Type H constructed like type G, except that the side pieces were nailed on with slope reversed at center, both sides of beam being alike. They cost the same as type G.

Beam #17 was placed in the machine with sidepieces sloping upward toward center. The lower 4×4 broke in tension at 5300#, the deflection then being about $5\frac{3}{4}$ inches. The sidepieces tended to split, cracks $\frac{3}{8}$ " wide showing between them in places. The 4×4 's were $\frac{1}{8}$ " apart at the supports caused by the tendency of the sidepieces to take a more nearly vertical position. When the load was removed, the deflection was only about $1\frac{1}{4}$ inches. The beam was then taken out of the machine, and forced back nearly straight, the fractured member was spiked together as well as possible. Then it was replaced in the machine with the other edge up, so that the sidepieces sloped downward toward the center. As badly as the sidepieces were split, the beam

held 7000* with a deflection of less than five inches before the lower member finally broke. In this position, the load tended to crowd the sidepieces together, and they in turn pressed the 4"x4"s together, thus increasing the friction between them.

Beam #18 was placed in the machine with the sidepieces sloping downward toward the center and the lower member broke at 8000*.

Type I shows an attempt made to resist the longitudinal shearing stresses, so evident in #15-#18, by putting in five $\frac{1}{2}$ " bolts between each load point and the adjacent support, together with the 1"x6" side pieces as in type H. Two 4"x6" pieces were used instead of three 4"x4"s and five 10d nails were driven through the 1"x6"s into each 4"x6". Previous tests showed that the beams should be placed in the machine with sidepieces sloping downward toward the center, and this type was so placed. The cost follows:-

2 men	2 hrs.	@ 55¢/hour	= \$1.10
2 pieces	4" x 6" x 16'-0"	@ \$28.00/M	= 1.80
5 "	1" x 6" x 16'-0"	@ 25.00/M	= 1.00
500 nails	10d	@ 2 1/2¢/#	= .20
20 bolts	1/2" x 13"		= <u>.40</u>
Total cost of one beam			= 4.50.

Beam #19. The first cracking was heard at 12000# and was caused by the nails splitting the side pieces. The lower member failed in tension at 17250#.

Beam #20 showed the first crack at 9000#, while the lower member broke at 14450#.

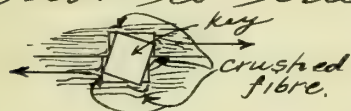
Type J. Since many of the beams had failed in tension near the center, this type was devised in order to strengthen the tension portion by having a 2" x 4" along the bottom, also to put in 2" x 4" diagonals to help resist the longitudinal shear. The beams were simply spiked together with 20d spikes driven in from each side. The cost of the beam is assumed as follows:-

2 men $\frac{3}{4}$ hour @ 55¢/hour = .41
 2 pieces 2"x12"x16'-0" @ \$26.00/M = 1.67
 1/4 " 2"x4"x16'-0" @ 26.00/M = .34
 80 spikes 20d @ 2 1/2¢/# = .14
 Total cost of one beam = 2.56.

Beam #21. The first crack appeared at 5000# and the beam broke near a knot at 9600#.

Beam #22. The first crack was heard at 4000# and the beam broke in two, at 12960#.

Type K. In this type six bolts and three cast iron keys were placed between the load points and the supports, these being assumed to transmit the longitudinal shear from one member to the other. The keys tended to rotate in their notches as in the figure so as not to transmit the stresses equally.



The cost follows:-

2 men 2 1/8 hours @ 55¢/hour = \$1.17
 2 pieces 6"x6"x16'-0" @ \$28.00/M = 2.69
 6 cast iron keys 2"x2"x8" @ 30¢/each = 1.80
 12 bolts with washers 1/2" dia x 13" long = .49
 Total cost for each beam = 6.15.

Beam #23. The first crack was heard at 6950#. At 14900# both members broke entirely off at one of the keys nearest a loaded point. Broken section showed that wood was of poor quality and brittle.

Beam #24. The first cracking was heard at 5000#. At 11900# the lower member broke at a key nearest a loaded point. The upper member was still sound.

Type L. This type has been used by the writer in actual practice and it was expected to show good results. Ten $\frac{1}{2}$ " bolts were put through the 2"x12"s and 2"x6" strut, while blocks were spiked in to prevent buckling of the 2"x12"s. It has been claimed that the strut carried, not only, a part of the longitudinal shear, but also, a part of the compression, as a truss. But it was proved that even a truss is no stronger than its weakest part, which is, in this case, the tension member. The cost was assumed to be as follows:-

2 men $\frac{7}{8}$ hours @ 55¢/hour = \$.50
 2 pieces 2" x 12" x 16'-0" @ \$26.00/M = 1.66
 1 " 2" x 6" x 18'-0" @ 26.00/M = .47
 30 spikes 20d @ $2\frac{1}{2}$ ¢/# = .03
 10 bolts, with washers, $\frac{1}{2}$ " x 5" = .34
 Total for one beam. = 3.00.

Beam #25 broke in tension at 12900#, after cracking at 11000#. This beam was taken apart afterward to see if the bolts had crushed the timbers at any point, and thus determine, if possible, the character of the stresses carried by the strut. No such crushing could be found at any place near a bolt.

Beam #26 showed the first crack at 10500#, and it broke at 14000#.

The average weight with 12% of moisture for type A is 32.8 #/cubic foot. Prof. J. B. Johnson, in his Materials of Construction, gives 32 pounds as the average weight of short leaf pine per cubic foot. The average ultimate tensile resistance of this material was 5660 #/sq. in. In the handbook for 1904 of the Southern Lumber Manufacturers Association, 13400 #/sq. in. is given as the average ultimate tensile strength of short leaf pine, or about $2\frac{3}{8}$ times as much.

	Average Type A.	Prof. J. B. Johnson.	Sou. Lmbr. Man. Assn.
E	1271000	1680000	1600000
R	5380	10100	9230

Type B gives a higher value for E than does type A.

	Type B.	Type A.
E	1670000	1271000
R	5190	5380

This shows what might be expected, that with selected lumber type B would give stronger and stiffer beams than type A, owing to the fact that a defect in

one member would not affect the other members of type B, but it might affect the whole beam of type A.

Types C E and F are not final tests, for both E and F might have been larger, had the ultimate strength of the beams been developed. Probably buckling was due to the nails slightly crushing into the wood, instead of holding it rigidly.

Types D G and H are very disappointing, because much better results were expected from them. Mr. T. M. Clark, in his book on Building Superintendence, mentions three small models like type G that developed 95% of the strength of a solid beam of similar dimensions, and that the ultimate resistance of the three models was almost exactly the same.

Owing to the fact that the side pieces of types G and H could not resist the longitudinal shear, it was thought that in type I the bolts with the side pieces would successfully bear this shear.

The following table will show that they did not do this.

	Load, $\Delta=52"$	R
Average, Type A.	5250	5380
Consider Type I as $\frac{2}{3}$ of Type A.	3500	3590
Actual Average, Type I	2590	3850

Types J and L may be compared with type B as follows;—

	Load, $\Delta=52"$	R
Average, Type B.	4370	5190
$\frac{2}{3}$ Type B, representing Types J & L.	2910	3460
Actual Average, Type J.	2920	3315
Actual Average, Type L.	2410	4150

The table does not prove the system of trussing beneficial. Comparing types J and L, J seems to be stiffer while L is ultimately the stronger, due no doubt to the fact that bolts were used in type L and bearing came directly on end surface of wood, while in all cases where nails are used the fibres are split or torn, and they bear against the wood like a sharp wedge.  Nail, Crushed fibre or open space.

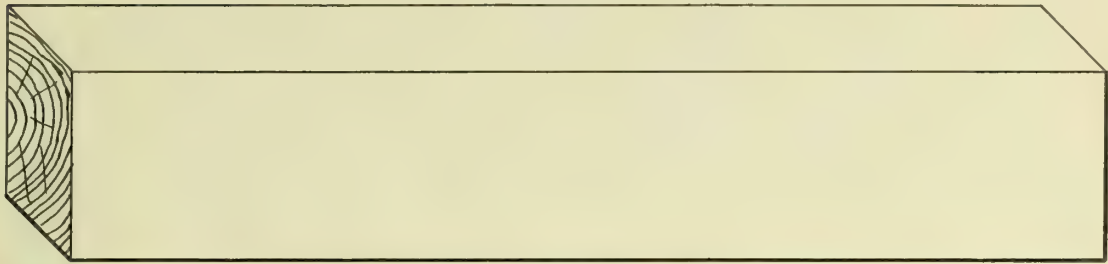
Type K was not a fair test, as the cast iron keys tended to rotate, thus crushing the beam instead

of resting on a fair bearing. Under better conditions this type would give far better results.

It seems that the best, cheapest and strongest beam is a solid beam straight grained and free from knots and cracks; that beams built up of pieces spiked side by side, as types B and C, are under ordinary conditions nearly as cheap and strong; that there are promising possibilities for types E and F; that the value of trussing, as in types J and L, is questionable; and that types D G H and I are entire failures.

DEFLECTIONS OF TYPE A.

Solid 6X12"-16'0" long.

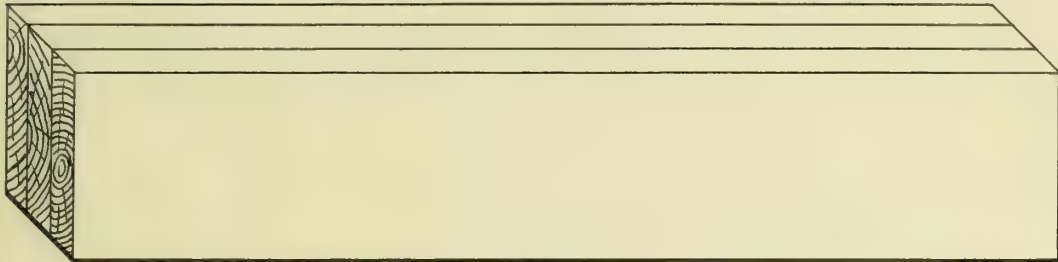


Load.	Beam #1.			Beam #2.			Beam #3.			Beam #4.		
	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.
0	0	0	0	0	0	0	0	0	0	0	0	0
1000 [#]	.09	.09	.12	.08	.09	.07	.09	.10	.08	.09	.10	.09
2000	.19	.21	.22	.16	.19	.15	.18	.21	.17	.17	.19	.17
3000	.29	.33	.33	.24	.28	.22	.26	.31	.26	.25	.28	.26
4000	.39	.45	.42	.32	.37	.30	.35	.41	.34	.33	.37	.34
5000	.50	.57	.52	.40	.46	.38	.43	.51	.43	.40	.46	.41
6000	.60	.69	.63	.48	.55	.45	.52	.61	.52	.48	.55	.49
7000	.70	.81	.73	.56	.64	.53	.60	.71	.60	.55	.64	.57
8000	.80	.93	.84	.64	.73	.61	.69	.81	.69	.63	.73	.65
9000	.90	1.05	.95	.72	.82	.69	.78	.91	.78	.70	.81	.72
10000	1.01	1.17	1.05	.79	.91	.76	.86	1.00	.87	.78	.90	.80
9000	.92	1.07	.96	.72	.84	.70	.79	.92	.79	.71	.82	.73
8000	.83	.95	.87	.64	.75	.63	.70	.82	.70	.64	.74	.66
7000	.73	.83	.77	.56	.66	.55	.62	.73	.62	.57	.66	.59
6000	.63	.72	.67	.50	.57	.48	.54	.63	.54	.49	.57	.50
5000	.53	.60	.57	.42	.48	.40	.45	.53	.45	.42	.49	.43
4000	.44	.49	.47	.36	.39	.32	.37	.43	.37	.35	.40	.35
3000	.33	.36	.37	.28	.30	.24	.28	.34	.28	.27	.31	.26
2000	.23	.24	.27	.20	.22	.16	.20	.24	.20	.19	.21	.18
1000	.13	.12	.16	.12	.12	.08	.11	.15	.11	.11	.12	.10
0	0	0	0	.06	.04	.02	0	.02	.03	.02	.02	0
1000	.10	.11	.16	.12	.12	.07	.12	.14	.12	.11	.12	.09
2000	.20	.23	.25	.21	.21	.14	.20	.24	.20	.19	.21	.18
3000	.30	.34	.35	.28	.30	.22	.29	.34	.29	.27	.30	.26
4000	.40	.46	.46	.36	.38	.29	.36	.43	.37	.34	.39	.34
5000	.50	.58	.56	.44	.47	.37	.44	.53	.46	.42	.48	.42
6000	.59	.69	.66	.52	.56	.44	.52	.62	.54	.49	.56	.50
7000	.69	.81	.76	.59	.65	.52	.61	.72	.62	.57	.65	.58
8000	.79	.92	.86	.67	.74	.59	.69	.82	.71	.64	.73	.65
9000	.89	1.04	.96	.75	.82	.67	.77	.91	.79	.71	.82	.73
10000	1.00	1.16	1.06	.82	.91	.75	.86	1.01	.88	.78	.90	.80
11000	1.10	1.28	1.17	.90	1.00	.83	.94	1.11	.97	.85	.99	.88
12000	1.20	1.39	1.27	.98	1.09	.90	1.03	1.21	1.06	.93	1.08	.95
13000	1.32	1.53	1.39	1.06	1.18	.98	1.12	1.31	1.14	1.01	1.18	1.03
14000	1.43	1.66	1.51	1.14	1.27	1.07	1.21	1.42	1.24	1.08	1.27	1.11
15000	1.55	1.85	1.64	1.22	1.36	1.14	1.31	1.54	1.35	1.16	1.36	1.19
16000	—	2.02	—	1.31	1.46	1.22	1.41	1.65	1.44	1.23	1.44	1.27
17000	—	2.20	—	1.39	1.55	1.30	1.50	1.76	1.54	1.31	1.53	1.35
18000	—	2.40	—	1.47	1.64	1.39	1.61	1.88	1.64	1.39	1.63	1.44
19000	—	2.65	—	1.57	1.77	1.49	1.74	2.04	—	1.48	1.73	1.53
20000	—	2.98	—	1.67	1.87	1.57	1.85	2.16	—	1.56	1.83	1.61
21000	—	3.50	—	—	1.98	—	—	2.31	—	1.65	1.94	1.71
22000	Failed at 21580 [#]			—	2.10	—	—	2.49	—	1.75	2.05	1.80
23000	—	—	—	—	2.23	—	—	2.69	—	—	2.18	—
24000	—	—	—	—	2.38	—	—	2.97	—	—	2.31	—
25000	—	—	—	—	—	—	—	—	—	—	—	—

Failed at 27880[#]. Failed at 26000[#]. Failed at 28150[#].

DEFLECTIONS OF TYPE B.

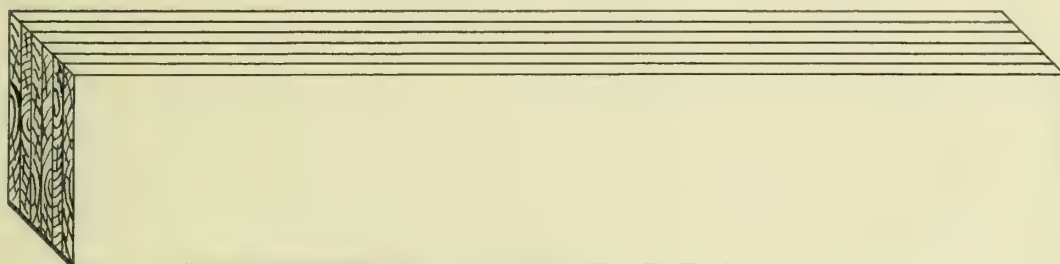
Three 2"X12"X16'0" spiked together.



Load.	Beam #5.			Beam #5a			Beam #6.		
	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.
0	0	0	0	0	0	0	0	0	0
1000*	.10	.10	.10	.12	.13	.11	.14	.16	.14
2000	.21	.20	.20	.23	.26	.22	.26	.28	.25
3000	.32	.30	.31	.34	.38	.33	.39	.40	.39
4000	.42	.40	.41	.43	.50	.44	.52	.54	.52
5000	.50	.51	.51	.54	.63	.55	.64	.68	.65
6000	.60	.61	.61	.65	.75	.66	.77	.82	.77
7000	.70	.72	.70	.75	.88	.77	.89	.95	.90
8000	.79	.83	.80	.86	1.00	.89	1.02	1.10	1.03
9000	.89	.94	.90	.97	1.13	1.01	1.16	1.25	1.18
10000	.99	1.05	.99	1.08	1.25	1.12	1.28	1.40	1.30
9000	.90	.95	.90	.98	1.14	1.02	1.20	1.28	1.21
8000	.82	.87	.83	.88	1.02	.92	1.08	1.16	1.10
7000	.73	.77	.74	.78	.90	.83	.96	1.02	.99
6000	.63	.66	.64	.67	.77	.71	.84	.89	.87
5000	.54	.55	.55	.57	.65	.60	.71	.75	.73
4000	.45	.44	.45	.46	.53	.49	.58	.60	.60
3000	.34	.33	.35	.35	.40	.38	.45	.48	.49
2000	.24	.22	.24	.24	.27	.27	.33	.32	.38
1000	.14	.10	.14	.13	.14	.16	.19	.19	.24
0	.02	0	.04	.01	.01	.04	.04	.06	.09
1000	.12	.09	.13	.13	.14	.16	.17	.18	.22
2000	.22	.20	.24	.23	.26	.27	.29	.29	.35
3000	.32	.30	.34	.34	.38	.38	.41	.44	.46
4000	.42	.41	.43	.45	.51	.49	.55	.58	.60
5000	.52	.54	.53	.55	.63	.60	.69	.72	.73
6000	.61	.63	.62	.65	.75	.70	.82	.87	.85
7000	.71	.74	.72	.76	.87	.81	.94	.99	.98
8000	.80	.85	.83	.87	1.00	.92	1.06	1.13	1.12
9000	.90	.97	.92	.98	1.12	1.03	1.19	1.30	1.24
10000	.99	1.06	1.00	1.08	1.24	1.14	1.31	1.41	1.37
11000	1.09	1.16	1.13	1.19	1.37	1.25	1.44	1.57	1.49
12000	1.18	1.27	1.22	1.31	1.50	1.37	Failed at 12000*		
13000	1.29	1.38	1.33	1.42	1.64	1.49			
14000	1.39	1.48	1.43	1.54	1.77	1.61			
15000	1.49	1.60	1.53	—	1.92	—			
16000	1.60	1.71	1.63	—	2.05	—			
17000	1.71	1.82	1.74	—	2.20	—			
18000	Failed at 17970*			—	2.35	—			
19000				—	2.52	—			
20000				Failed at 21550*					

DEFLECTIONS OF TYPE C.

Six 1"X12"X16'0" nailed together.

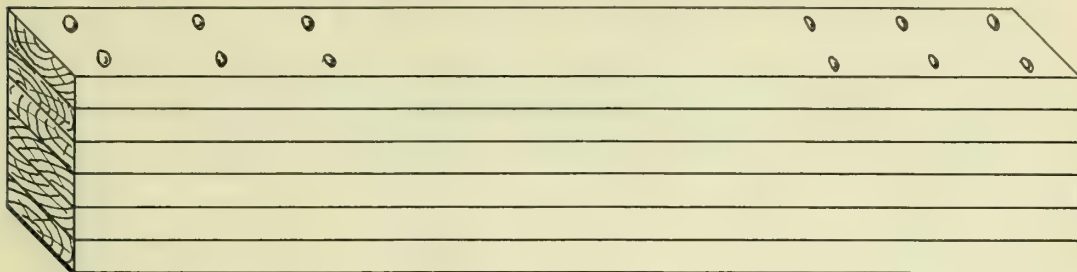


Load.	Beam #7.			Beam #8.		
	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.	N $\frac{1}{2}$.	Cent.	S $\frac{1}{2}$.
0	0	0	0	0	0	0
1000 #	.10"	.08"	.11"	.10"	.10"	.10"
2000	.19	.18	.20	.19	.20	.19
3000	.27	.24	.28	.27	.30	.29
4000	.35	.33	.37	.37	.40	.39
5000	.44	.43	.46	.46	.52	.48
6000	.52	.53	.54	.56	.62	.57
7000	.60	.63	.62	.65	.73	.67
8000	.69	.71	.70	.74	.83	.77
9000	.76	.80	.79	.83	.92	.86
10000	.84	.87	.87	.92	1.02	.94
9000	.79	.81	.81	.83	.92	.85
8000	.70	.72	.72	.75	.83	.78
7000	.62	.63	.65	.66	.74	.69
6000	.55	.56	.57	.57	.65	.60
5000	.47	.47	.48	.48	.56	.51
4000	.39	.38	.41	.39	.45	.42
3000	.31	.29	.32	.30	.34	.30
2000	.23	.21	.24	.21	.23	.18
1000	.15	.11	.14	.11	.13	.12
0	.06	.04	.06	.02	.02	.03
1000	.14	.12	.14	.11	.12	.12
2000	.21	.18	.23	.20	.22	.21
3000	.29	.28	.31	.28	.32	.30
4000	.37	.37	.39	.38	.43	.39
5000	.45	.46	.47	.47	.53	.49
6000	.53	.56	.55	.56	.63	.58
7000	.61	.63	.64	.65	.73	.67
8000	.69	.72	.72	.74	.84	.76
9000	.77	.81	.80	.84	.94	.85
10000	.84	.88	.87	.93	1.03	.94
11000	.92	.96	.96	1.01	1.13	1.02
12000	1.00	1.05	1.04	1.11	1.23	1.12
13000	1.09	1.12	1.13	1.20	1.32	1.21
14000	1.21	1.21	1.22	1.30	1.42	1.30
15000	Failed at 14400*			1.40	1.55	1.40
16000				1.54	1.71	1.53
17000				Failed at 16310*		
18000						
19000						
20000						

DEFLECTIONS OF TYPES D, G & H.

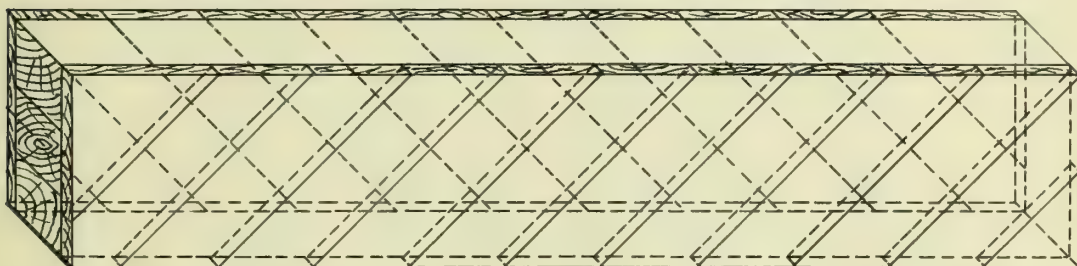
Beams #9 & #10.

Six 2"X6"X16'0" bolted and spiked together.



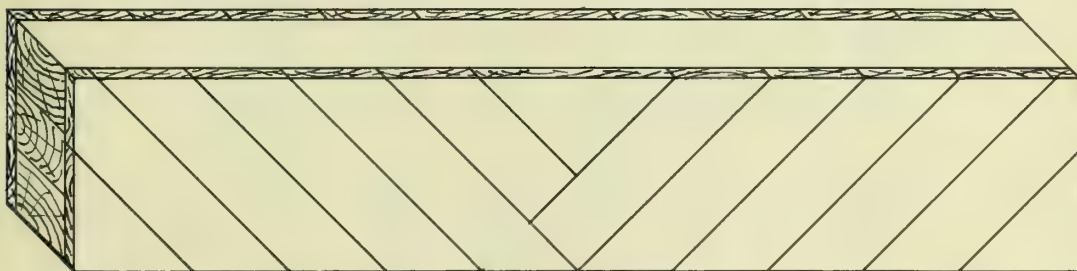
Beams #15 & #16.

Three 4"X4"X16'0" & 1"X6's nailed together.



Beams #17 & #18.

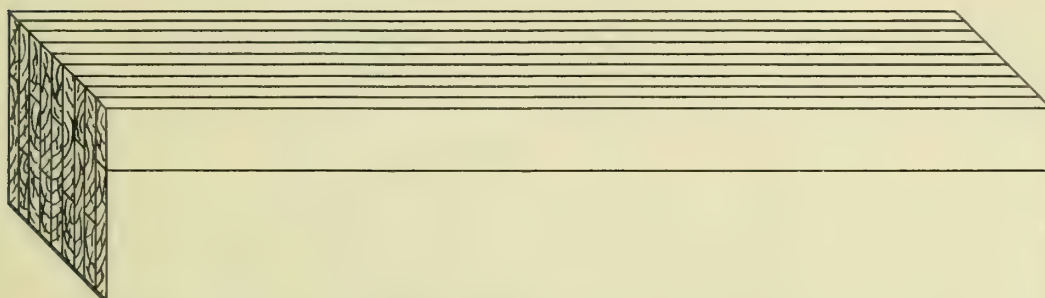
Three 4"x4"x16'0" & 1"x6's nailed together.



	#9.	#10.	#15.	#16.	#17.	#18.
Load.	Cent.	Cent.	Cent.	Cent.	Cent.	Cent.
0	0	0	0	0	0	0
1000#	.16"	.37"	.44"	.44"	.81"	.65"
2000	1.04	1.24	1.06	1.00	1.25	1.30
3000	2.16	2.45	1.75	1.69	1.96	1.92
4000	3.85	3.95	2.63	2.56	2.69	2.51
5000	5.60	5.45	3.69	3.69	3.57	3.44
6000	7.10	6.20	5.00	4.90	4.69	5.00
7000	9.00	8.70	6.69	6.38	Failed.	5.88
8000	10.35	10.20	Failed at	Failed at		Failed.
9000	Fiber crushed at bolt		7900#	7410#		
10000	& bolts bent.					

DEFLECTIONS OF TYPE E.

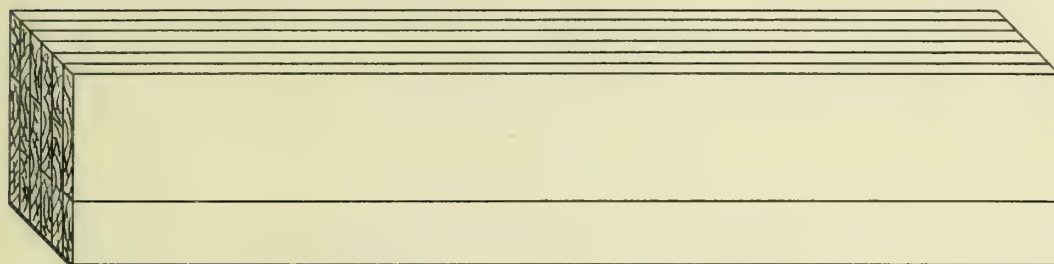
Nine 1X8X16'0" and 1X4X16'0" nailed together.



Load.	Beam #11.			Beam #12.		
	N $\frac{1}{2}$	Cent.	S $\frac{1}{2}$	N $\frac{1}{2}$	Cent.	S $\frac{1}{2}$
0	0	0	0	0	0	0
1000 #	.09"	.10"	.09"	.09"	.10"	.09"
2000	.18	.20	.18	.18	.19	.18
3000	.27	.31	.27	.28	.30	.28
4000	.36	.42	.37	.37	.40	.38
5000	.46	.53	.47	.46	.51	.47
6000	.56	.66	.58	.56	.63	.58
7000	.66	.78	.68	.66	.74	.68
8000	.76	.90	.78	.77	.86	.79
9000	.87	1.02	.89	.88	1.00	.91
10000	1.01	1.19	1.04	.98	1.15	1.03
9000	.95	1.12	.98	.94	1.08	.98
8000	.87	1.02	.89	.88	1.00	.91
7000	.80	.93	.81	.78	.88	.81
6000	.72	.84	.73	.70	.79	.74
5000	.63	.73	.64	.62	.69	.66
4000	.60	.64	.56	.54	.60	.57
3000	.50	.53	.46	.45	.49	.48
2000	.39	.40	.35	.35	.39	.38
1000	.30	.29	.25	.25	.28	.28
0	.18	.16	.14	.15	.16	.18
1000	.26	.25	.22	.23	.25	.26
2000	.35	.35	.31	.31	.34	.35
3000	.44	.45	.39	.40	.44	.43
4000	.52	.55	.49	.49	.55	.52
5000	.61	.65	.59	.57	.64	.61
6000	.70	.75	.68	.66	.74	.70
7000	.79	.87	.77	.75	.84	.79
8000	.89	.98	.87	.84	.94	.88
9000	.99	1.09	.97	.93	1.04	.98
10000	1.09	1.21	1.07	1.03	1.14	1.07
11000	1.20	1.34	1.18	1.13	1.26	1.18
12000	1.33	1.50	1.32	1.25	1.40	1.31
13000	1.48	1.68	1.47	1.39	1.56	1.45
14000	1.64	1.86	1.63	—	1.75	—
15000	—	2.06	—	—	1.94	—
16000	—	2.36	—	—	2.15	—
17000	—	2.65	—	—	2.38	—
18000	—	3.18	—	Failed at 18200 [#]		
19000	Failed at 18100 [#]					

DEFLECTIONS OF TYPE F.

Six 1X8X16'0" and 1X4X16'0" nailed together.

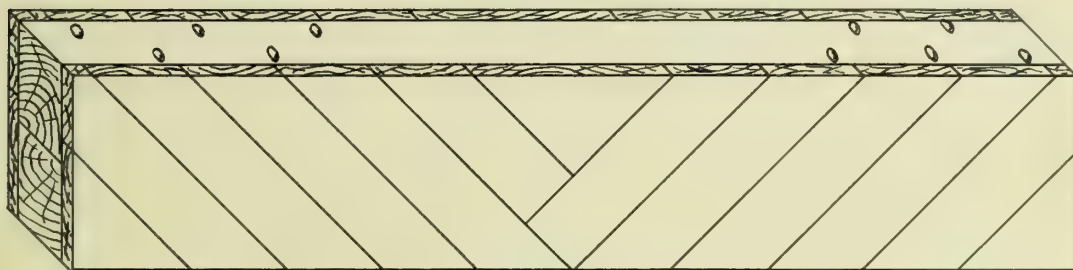


Load	Beam #13.			Beam #14.		
	N $\frac{1}{2}$	Cent.	S $\frac{1}{2}$	N $\frac{1}{2}$	Cent.	S $\frac{1}{2}$
0	0	0	0	0	0	0
1000#	.12"	.14"	.12"	.15"	.17"	.14"
2000	.26	.29	.26	.28	.32	.28
3000	.39	.44	.39	.43	.49	.43
4000	.54	.61	.54	.58	.66	.57
5000	.68	.77	.67	.74	.84	.73
6000	.82	.94	.82	.91	1.03	.89
7000	.98	1.13	.98	1.09	1.23	1.07
8000	1.14	1.31	1.14	1.29	1.45	1.26
9000	1.31	1.50	1.31	1.51	1.68	1.46
10000	1.50	1.81	1.51	—	—	—
9000	—	—	—	—	—	—
8000	1.30	1.58	1.30	1.43	1.60	1.38
7000	—	—	—	—	—	—
6000	1.09	1.43	1.10	1.17	1.30	1.12
5000	—	—	—	.95	1.17	1.00
4000	.81	1.04	.81	.81	1.01	.87
3000	—	—	—	.66	.84	.72
2000	—	—	—	.49	.65	.55
1000	.37	.52	.36	.33	.47	.40
0	.18	.31	.17	.13	.29	.13
1000	.29	.44	.29	.26	.44	.26
2000	.42	.58	.41	.39	.59	.39
3000	.56	.74	.55	.54	.76	.54
4000	.70	.90	.69	.70	.94	.70
5000	.85	1.07	.84	.86	1.11	.85
6000	.99	1.21	.97	1.01	1.29	1.01
7000	1.14	1.38	1.12	1.16	1.46	1.16
8000	1.28	1.55	1.26	1.31	1.63	1.31
9000	1.41	1.71	1.40	1.47	1.80	1.46
10000	1.55	1.87	1.54	—	2.09	—
11000	Failed at 10300#			Failed at 10500#		
12000						

DEFLECTIONS OF TYPES I & J.

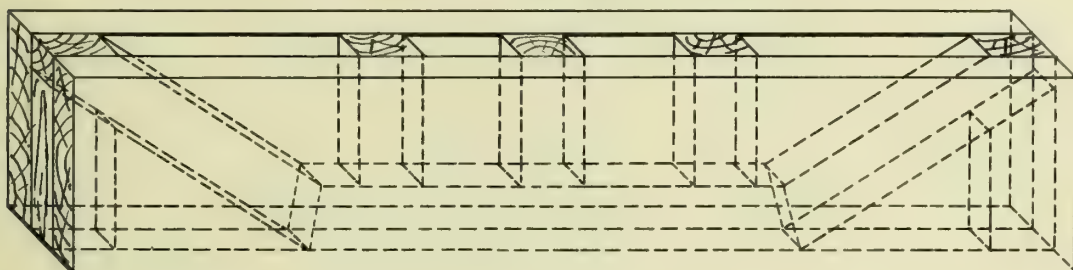
Beams #19 & #20.

Two 6"x6"x16'-0" bolted & 1"x6"s nailed together.



Beams #21 & #22

Two 2"x12"x16'0" & 2"x4's spiked together.



Load	Beam #19			Beam #20			Beam #21			Beam #22		
	N $\frac{1}{2}$	Cent	S $\frac{1}{2}$	N $\frac{1}{2}$	Cent	S $\frac{1}{2}$	N $\frac{1}{2}$	Cent	S $\frac{1}{2}$	N $\frac{1}{2}$	Cent	S $\frac{1}{2}$
0	0	0	0	0	0	0	0	0	0	0	0	0
1000#	14"	15"	14"	19"	21"	19"	16"	17"	16"	16"	14"	20
2000	30	34	30	39	45	40	34	36	32	32	33	36
3000	47	54	48	60	69	62	51	55	49	48	52	53
4000	66	75	67	84	96	85	68	75	66	65	71	69
5000	84	98	87	107	123	110	85	94	82	82	91	86
6000	105	122	108	133	151	136	103	116	100	99	110	104
7000	126	146	129	160	181	162	121	135	117	117	130	121
8000	150	173	154	187	213	189	140	156	135	135	150	139
9000	182	210	186	220	251	223	161	179	155	153	171	158
10000	207	239	212	242	277	246	Failed at 9600*			172	192	177
11000	241	277	246	261	298	265				—	216	—
12000	—	297	—	—	346	—				—	240	—
13000	—	341	—	—	402	—				Failed at 12960*		
14000	—	384	—	—	474	—						
15000	—	432	—	Failed at 14450*								
16000	—	483	—									
17000	Failed at 17250*											

Beam No.	Actual Size.	Ultimate Load.	Max'm Defl.	Mod. of Elast. $\Delta = .52"$	Long. Rupture Shear.	Moisture %.	Cost per Beam.	Cost per 1000' Cent'd.	Manner of Failure.
1.	6 $\frac{1}{8}$ x 12 $\frac{1}{8}$	21580	3.50	4580	4460	35.8	2.88	629	Tension.
2.	6 $\frac{1}{8}$ x 12 $\frac{1}{8}$	27880	2.55	5660	5640	31.9	"	509	"
3.	6 x 12 $\frac{1}{8}$	26000	3.25	5100	5490	17.8	"	565	Compression.
4.	6 x 12 $\frac{1}{8}$	28150	3.60	5670	5940	17.9	"	508	Tension.
5.	4 $\frac{3}{8}$ x 11	17970	1.82	5100	5670	15.6	2.96	580	Shear.
5a.	4 $\frac{3}{8}$ x 11 $\frac{1}{8}$	21550	2.52	4150	6350	17.3	"	713	"
6.	4 $\frac{3}{8}$ x 11 $\frac{1}{8}$	12000	1.57	3850	3540	14.7	"	770	Tension.
7.	4 $\frac{3}{8}$ x 12 $\frac{1}{8}$	14400	1.21	5900	3590	14.2	3.20	543	Buckled.
8.	4 $\frac{3}{8}$ x 12 $\frac{1}{8}$	16310	1.71	5000	3980	13.7	"	640	"
9.	5 $\frac{1}{8}$ x 9 $\frac{1}{4}$	8000	10.35	1400	2850	—	3.64	2885	Bolts bent.
10.	5 $\frac{1}{8}$ x 9 $\frac{1}{4}$	8000	10.20	1160	2850	—	"	3138	"
11.	7 $\frac{1}{2}$ x 12 $\frac{1}{8}$	18100	3.18	4900	2990	24.7	5.13	1047	Buckled.
12.	7 $\frac{1}{2}$ x 12	18200	2.38	5080	3130	24.3	"	1010	"
13.	5 x 12	10300	1.87	3470	2730	15.7	3.42	986	"
14.	5 x 12 $\frac{1}{8}$	10500	2.09	3180	2730	14.9	"	1075	"
15.	5 $\frac{1}{4}$ x 11	7900	6.69	1130	2310	—	3.38	2991	Tension.
16.	5 $\frac{1}{4}$ x 11	7410	6.38	1140	2170	—	"	2965	"
17.	5 $\frac{1}{4}$ x 11	7000	4.69	640	2050	—	"	5281	"
18.	5 $\frac{1}{4}$ x 11	8000	5.88	800	2340	—	"	4225	"
19.	5 $\frac{3}{8}$ x 12	17250	5.53	2900	4150	14.4	4.50	1552	"
20.	5 $\frac{3}{8}$ x 12	14450	4.74	2280	3550	15.3	"	1974	"
21.	4 $\frac{3}{8}$ x 11 $\frac{1}{8}$	9600	1.79	2840	2890	21.5	2.56	901	"
22.	4 $\frac{3}{8}$ x 11 $\frac{1}{8}$	12960	2.40	3000	3740	19.1	"	853	"
23.	5 $\frac{1}{2}$ x 11 $\frac{1}{8}$	14900	3.29	2390	3900	19.4	6.15	2573	"
24.	5 $\frac{1}{2}$ x 11 $\frac{1}{8}$	11900	1.90	3120	2910	18.2	"	1971	"
25.	4 $\frac{3}{8}$ x 11 $\frac{1}{8}$	12900	2.47	2400	3980	13.7	3.00	1250	"
26.	4 $\frac{3}{8}$ x 11 $\frac{1}{8}$	14000	2.75	2420	4320	—	"	1240	"

Safe loads in pounds uniformly distributed for type A.

$$W = \frac{384 E I \Delta}{5 l^3}$$

$$W = \text{safe load in pounds.}$$

$$E = 1271000$$

$$b = \text{breadth of beam in inches.}$$

$$I = \frac{bd^3}{12}$$

$$W = 24400 \frac{bd^3}{l^2}$$

$$d = \text{depth of beam in inches.}$$

$$l = \text{length of beam in inches.}$$

$$\Delta = .0031$$

L'	d = 9 5/8"			d = 11 5/8"			d = 13 5/8"			d = 15 5/8"			
	b = 3 5/8"	5 5/8"	7 5/8"	9 5/8"	3 5/8"	5 5/8"	7 5/8"	9 5/8"	5 5/8"	7 5/8"	9 5/8"	11 5/8"	13 5/8"
10	5500	8500	11500	14500	9600	15000	20300	25600					
12	3800	5900	8000	10100	6700	10400	14100	17800	16900	22700	28700	34600	43200
14	2800	4300	5900	7400	4900	7600	10400	13100	12300	16700	21000	25400	31700
16	2100	3300	4500	5700	3800	5900	7900	10000	9400	12800	16100	19600	24300
18	1700	2600	3600	4500	3000	4600	6300	7900	7400	10100	12700	15400	19200
20	1400	2100	2900	3600	2400	3700	5100	6400	6000	8200	10300	12500	15500
22					2000	3100	4200	5300	5000	6800	8500	10300	12800
24									4200	5900	7200	8700	10800
26												7300	9200

$$R = 5370.$$

$$W = \frac{8 R I}{l e}$$

$$I = \frac{bd^3}{6}$$

$$W = 1790 \frac{bd^3}{l}$$

$$\text{Factor of safety} = 4.$$

L'	d = 9 5/8"			d = 11 5/8"			d = 13 5/8"			d = 15 5/8"		
	b = 3 5/8"	5 5/8"	7 5/8"	9 5/8"	7 5/8"	5 5/8"	9 5/8"	7 5/8"	5 5/8"	9 5/8"	7 5/8"	5 5/8"
10	5000	7800	10600	13300	7300	11400	15400	19400				
12	4200	6500	8800	11100	6100	9500	12800	16200	13000	17700	22200	26900
14	3600	5600	7500	9500	5200	8100	11000	13900	11200	15100	19100	23000
16	3100	4900	6600	8300	4600	7100	9600	12100	9800	13200	16700	20100
18	2800	4300	5900	7400	4100	6300	8600	10800	8700	11800	14900	18000
20	2500	3900	5300	6700	3700	5700	7700	9700	7800	10600	13300	16000
22					3300	5200	7000	8800	7100	9600	12100	14600
24									6500	8800	11100	13400
26												10700

Safe load in pounds uniformly distributed for type B.

$$W = \frac{384 E I \Delta}{5 l^3} \quad W = \text{safe load in pounds.} \quad E = 1670000.$$

$b = \text{breadth of beam in inches.}$

$$I = \frac{b d^3}{12}$$

$d = \text{depth of beam in inches.}$

$l = \text{length of beam in inches.} \quad \Delta = .0031.$

L'	d = 9 1/2"				d = 11 1/2"				d = 13 1/2"				d = 15 1/2"			
	4 3/4"	6 1/2"	8 1/2"	4 3/4"	6 1/2"	8 1/2"	9 1/2"	6 1/2"	8 1/2"	9 1/2"	11 1/2"	8 1/2"	9 1/2"	11 1/2"	13"	13"
10	6200	9700	12900	16100	17000	22700	28400	34000								
12	4300	6700	9000	11200	11800	15800	19700	23700	25400	31700	38100	44500	48000	57600	67200	76600
14	3200	4900	6600	8200	8700	11600	14500	17400	18700	23300	28000	32700	35300	42300	49400	56300
16	2400	3800	5000	6300	6600	8900	11100	13300	14300	17900	21400	25000	27000	32400	37700	43100
18	1900	3000	4000	5000	5200	7000	8700	10500	11300	14100	16900	19700	21300	25600	29800	34000
20	1600	2400	3200	4000	4300	5700	7100	8500	9100	11400	13700	16000	17300	20700	24200	27600
22					3500	4700	5900	7000	7600	9400	11300	13200	14300	17100	20000	22800
24									6400	7900	9500	11100	12000	14400	16800	19200
26												10200	12300	14300	16300	

$$W = \frac{8 R I}{l e} \quad R = 5190.$$

$$I = \frac{b d^2}{e}$$

$$W = 1730 \frac{b d^3}{l}$$

Factor of safety = 4.

L'	d = 9 1/2"				d = 11 1/2"				d = 13 1/2"				d = 15 1/2"			
	4 3/4"	6 1/2"	8 1/2"	4 3/4"	6 1/2"	8 1/2"	9 1/2"	6 1/2"	8 1/2"	9 1/2"	11 1/2"	8 1/2"	9 1/2"	11 1/2"	13"	13"
10	4200	6500	8700	10900	9500	12700	15900	19000								
12	3500	5400	7300	9100	7900	10600	13200	15900	14500	18200	21800	25400	23900	28700	33400	38200
14	3000	4700	6200	7800	6800	9000	11300	13600	12400	15500	18600	21700	20500	24600	28700	32800
16	2600	4100	5400	6800	5900	7900	9900	11900	10900	13600	16300	19100	17900	21500	25100	28700
18	2300	3600	4800	6000	5300	7000	8800	10600	9700	12100	14500	16900	15900	19100	22300	25500
20	2100	3300	4400	5400	4800	6300	7900	9500	8700	10900	13100	15200	14300	16800	20000	23000
22					4300	5800	7200	8600	7900	9900	11900	13800	13000	15600	18200	20800
24									7300	9100	10900	12700	12000	14300	16700	19100
26												11000	13200	15400	17700	

Safe loads in pounds uniformly distributed for types E & F.

$$E = 1000000.$$

$$W = \frac{384EI\Delta}{5l^3}$$

$$W = 19200 \frac{bd^3}{l^2}$$

$$b = \text{breadth of beam in inches.}$$

$$d = \text{depth of beam in inches.}$$

$$l = \text{length of beam in inches.}$$

$$I = \frac{bd^3}{12}$$

$$\Delta = .0031.$$

	d=12"			d=14"			d=16"			d=18"			d=20"		
L'	b=4 1/2"	6 1/2"	8 1/2"	9 3/4"	11 3/8"	13"	14 5/8"	16"	18"	20"	22"	24"	26"	28"	30"
10	11300	15000	18700	22500	26300	30100	33900	37700	41500	45300	49100	52900	56700	60500	64300
12	7800	10400	13000	15600	18200	20800	23400	26000	28600	31200	33800	36400	39000	41600	44200
14	5700	7600	9600	11500	13500	15500	17500	19500	21500	23500	25500	27500	29500	31500	33500
16	4400	5900	7200	8800	10400	12000	13600	15200	16800	18400	20000	21600	23200	24800	26400
18	3500	4600	5800	7000	8200	9400	10600	11800	13000	14200	15400	16600	17800	19000	20200
20	2800	3700	4700	5600	6600	7600	8600	9600	10600	11600	12600	13600	14600	15600	16600
22					6100										
24															
26															

$$W = \frac{8RI}{le}$$

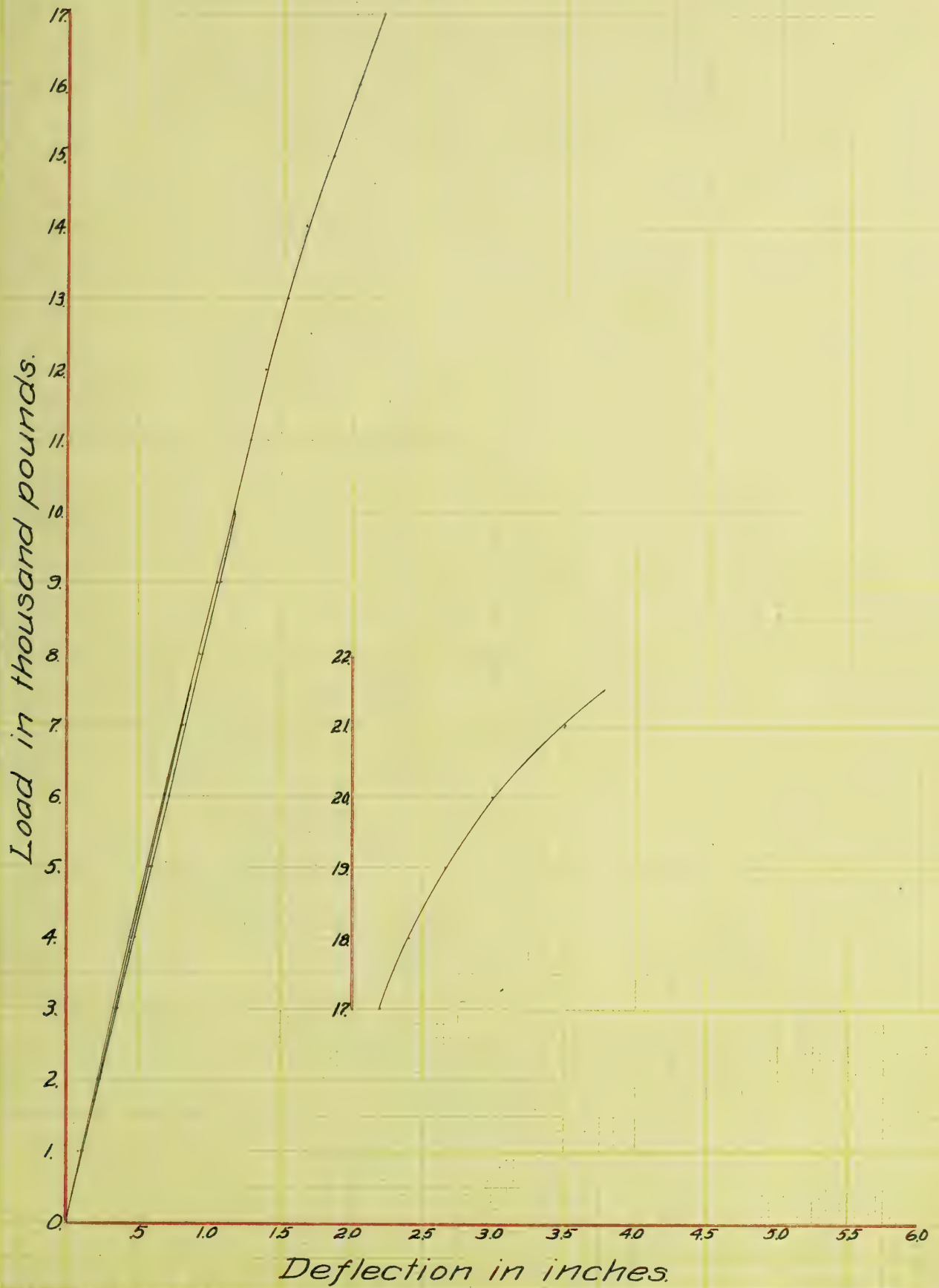
$$R = 2880.$$

$$I = \frac{bd^3}{12}$$

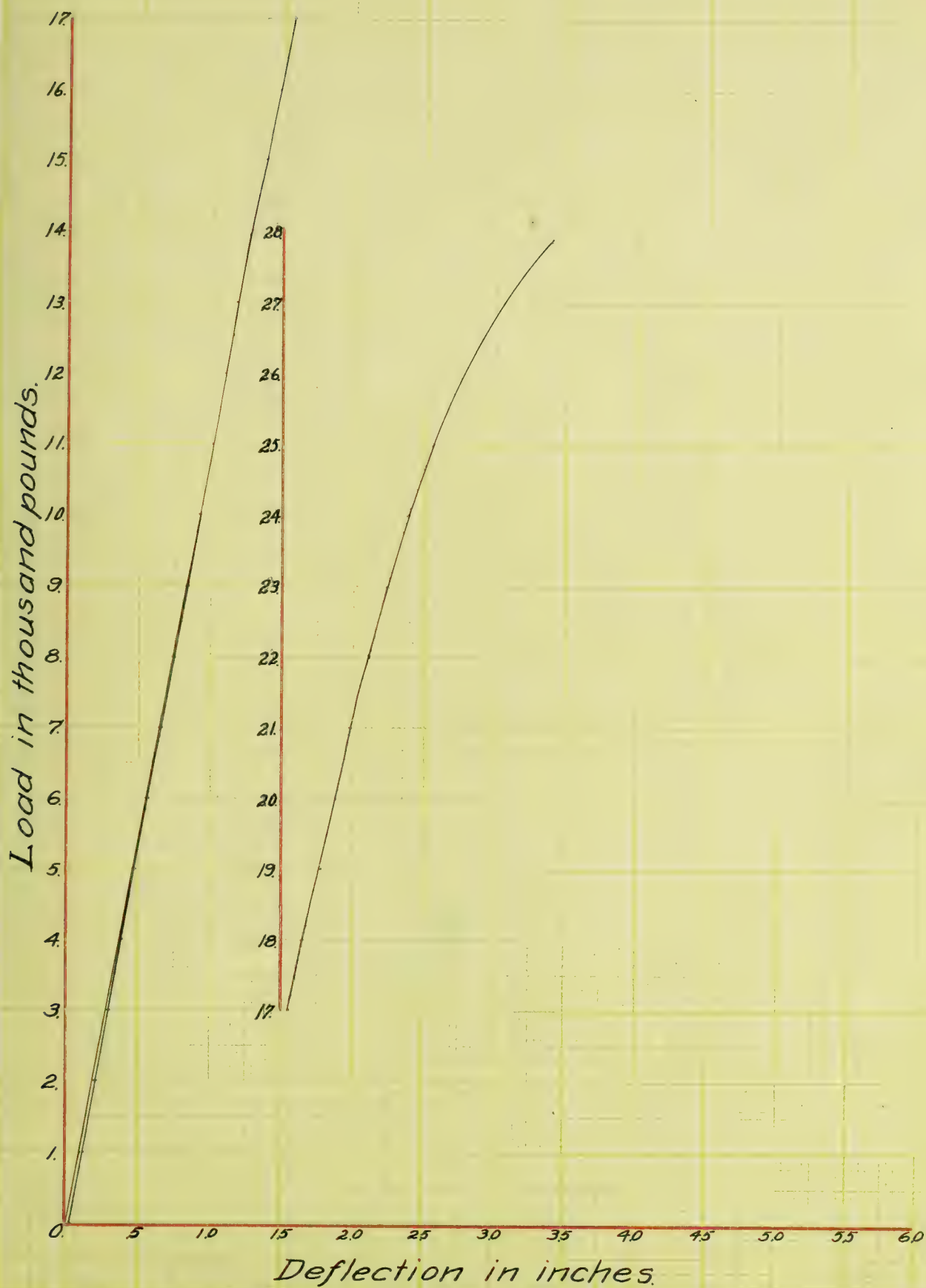
$$W = 960 \frac{bd^3}{l}$$

$$\text{Factor of safety} = 4.$$

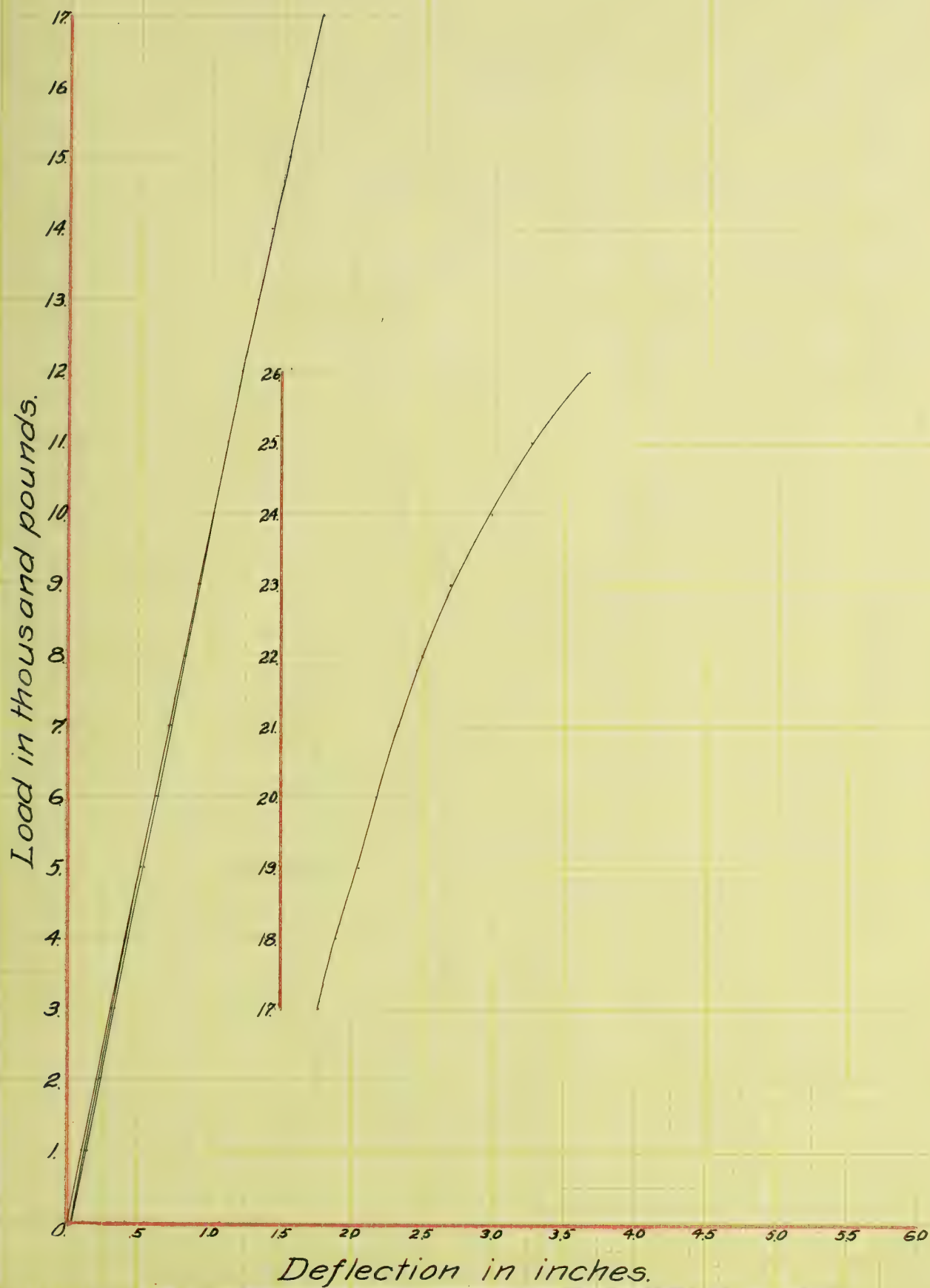
	d=12"			d=14"			d=16"			d=18"			d=20"		
L'	b=4 1/2"	6 1/2"	8 1/2"	9 3/4"	11 3/8"	13"	14 5/8"	16"	18"	20"	22"	24"	26"	28"	30"
10	5610	7480	9370	11210	13050	14890	16730	18570	20410	22250	24090	25930	27770	29610	31450
12	4680	6240	7800	9350	10900	12450	14000	15550	17100	18650	20200	21750	23300	24850	26400
14	4010	5350	6680	8010	9350	10680	12010	13340	14670	16000	17330	18660	19990	21320	22650
16	3510	4680	5850	7010	8170	9340	10500	11660	12820	13980	15140	16300	17460	18620	19780
18	3120	4160	5200	6240	7280	8320	9360	10400	11440	12480	13520	14560	15600	16640	17680
20	2810	3740	4670	5610	6540	7470	8400	9330	10260	11190	12120	13050	13980	14910	15840
22					5780										
24															
26															

Beam #1.

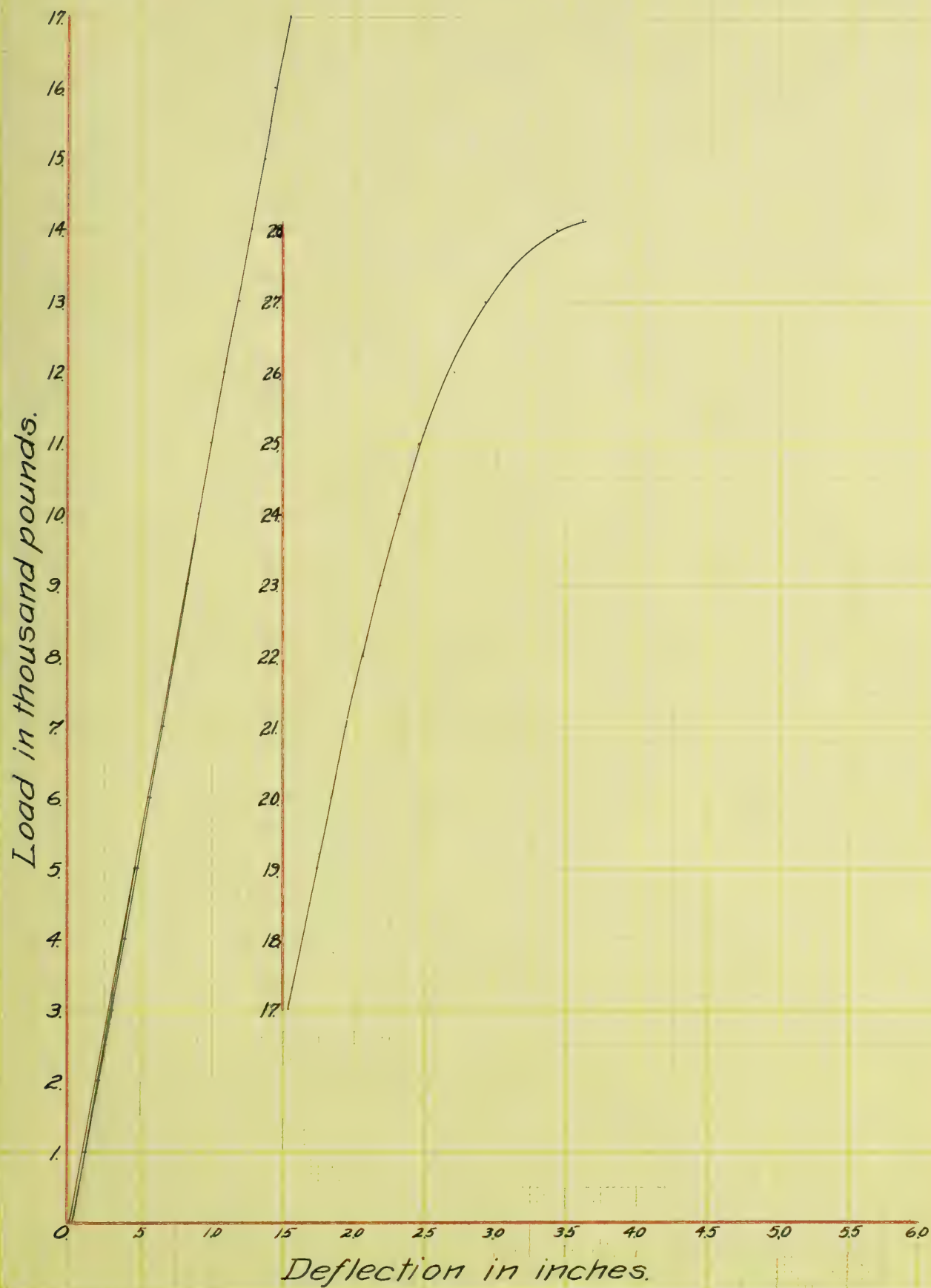
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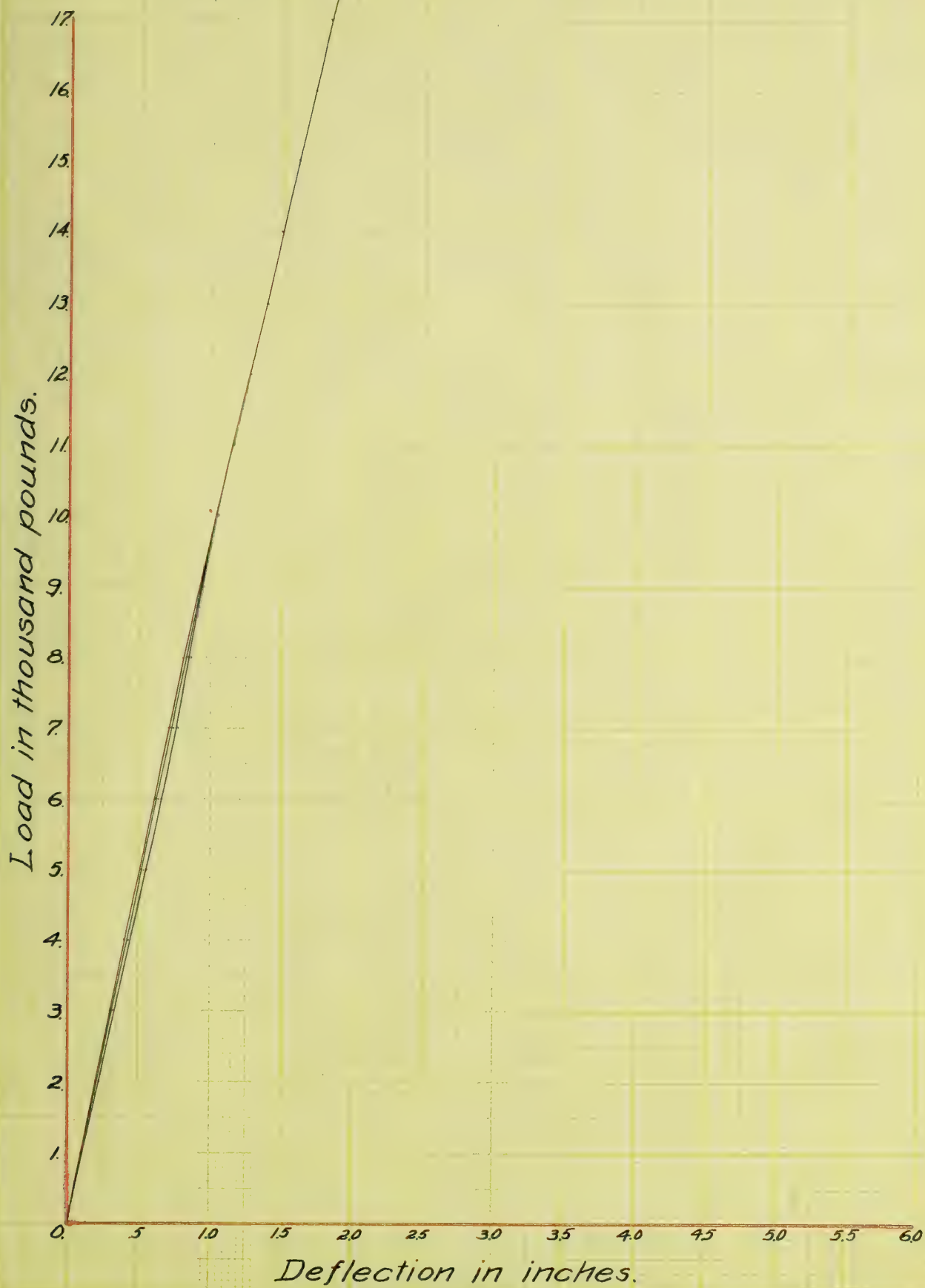


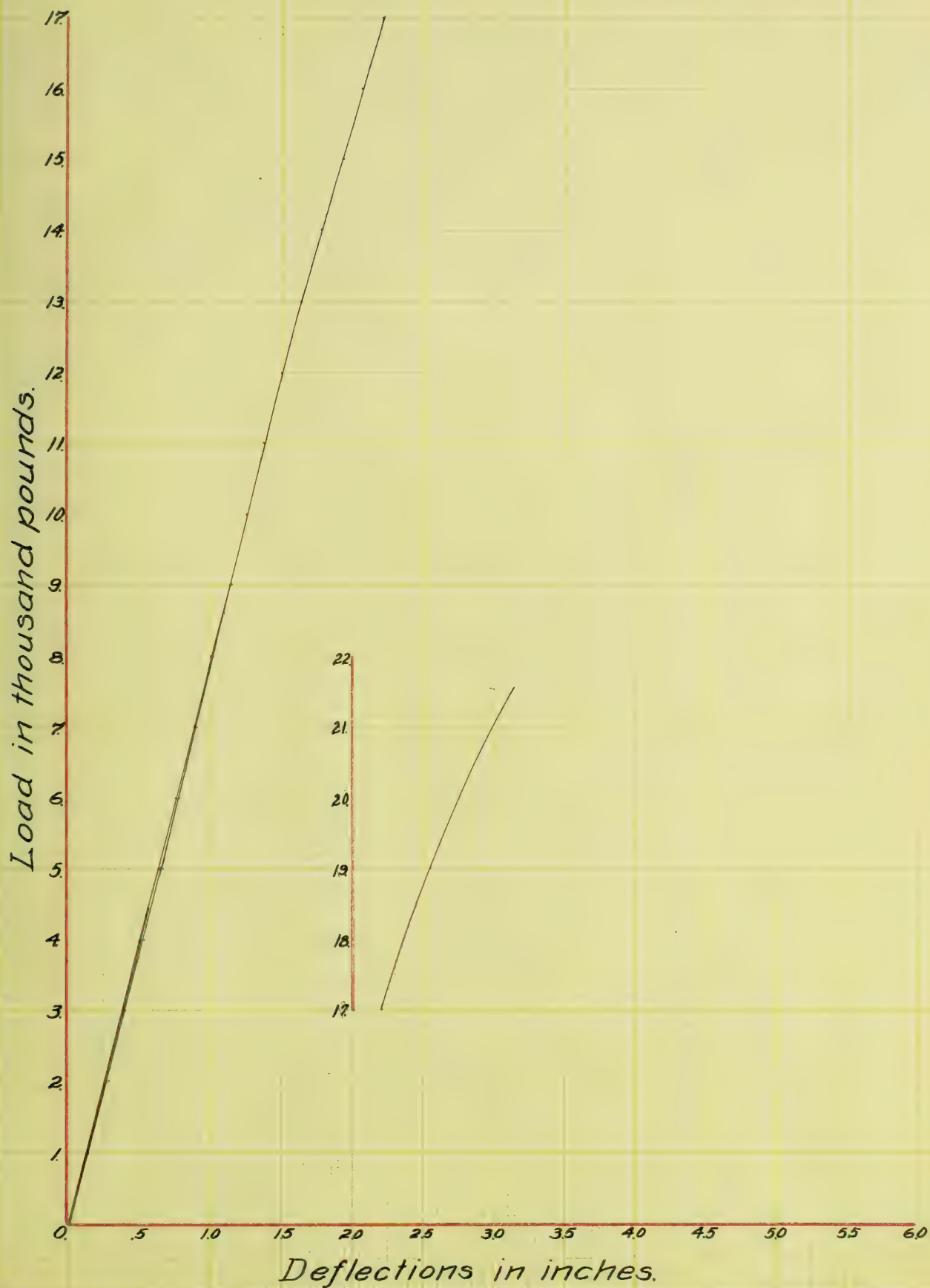
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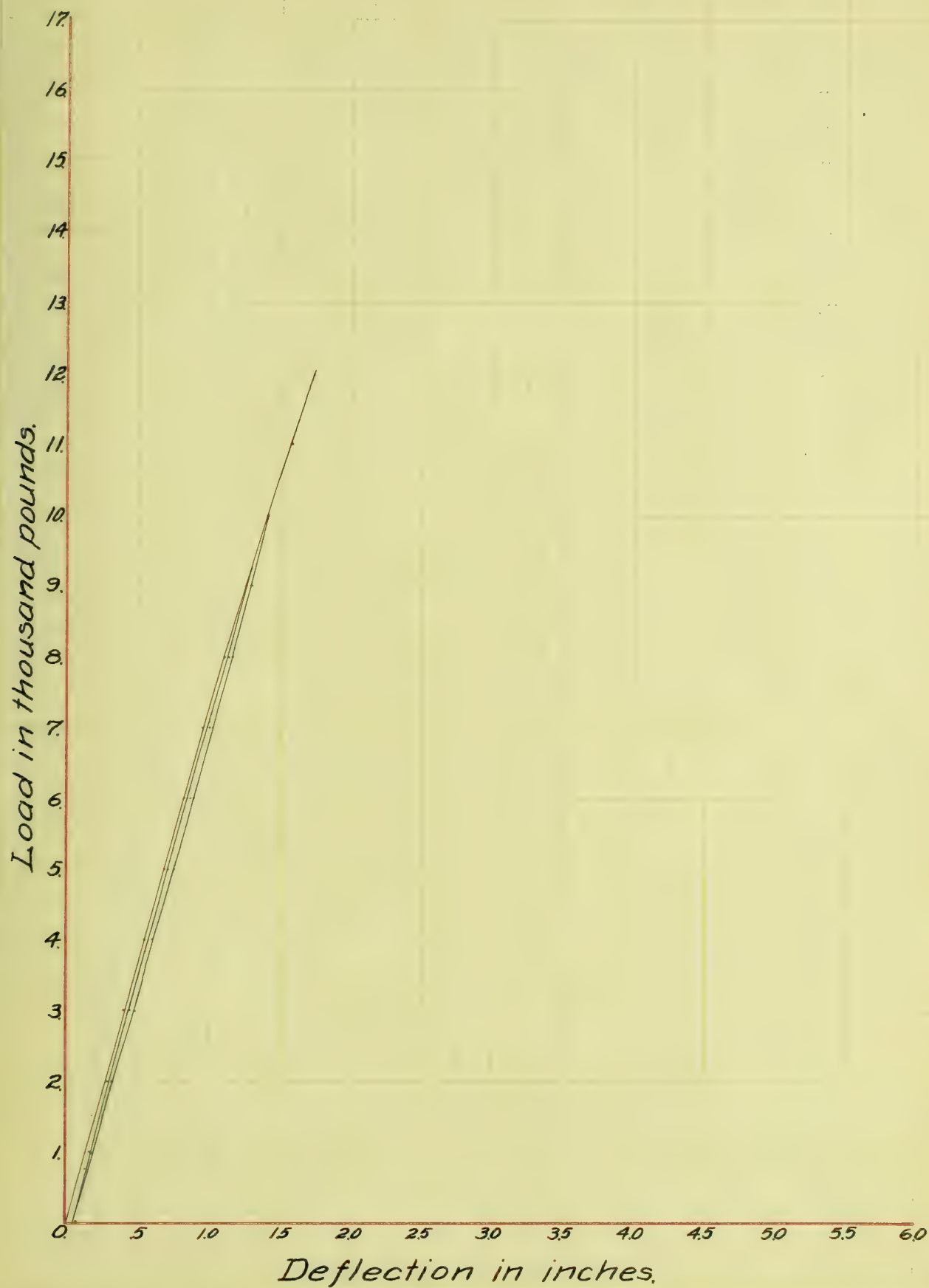


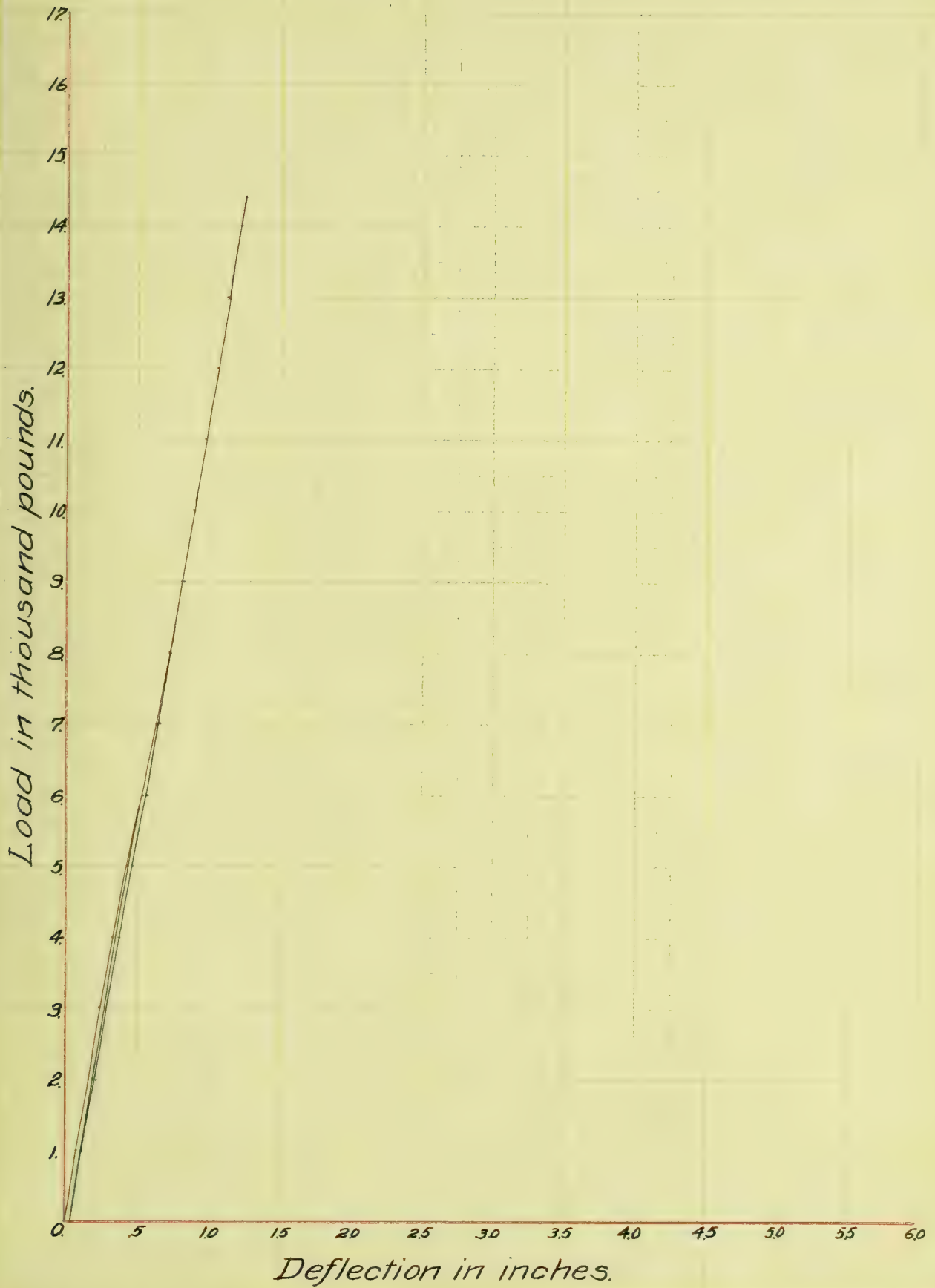
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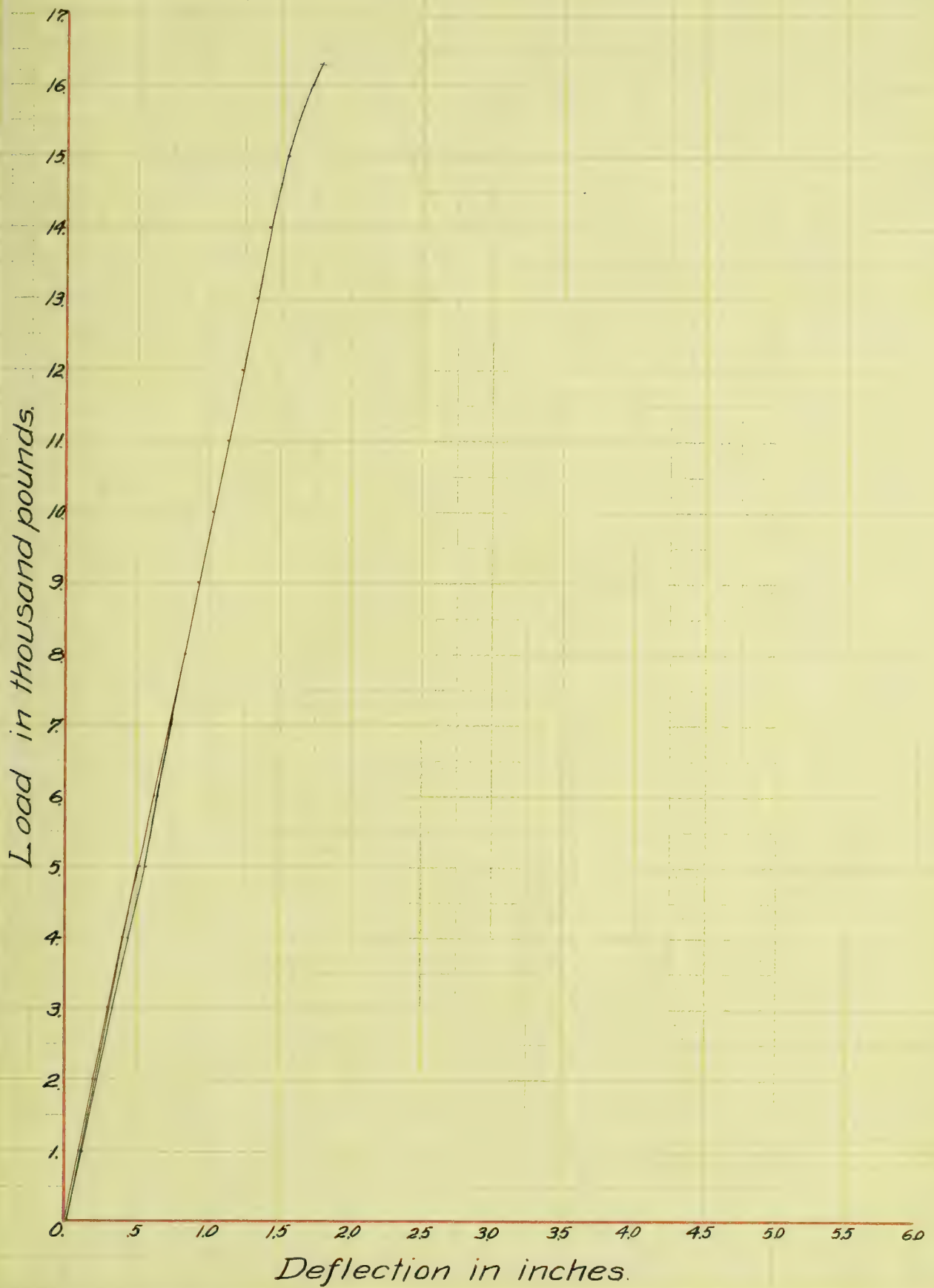


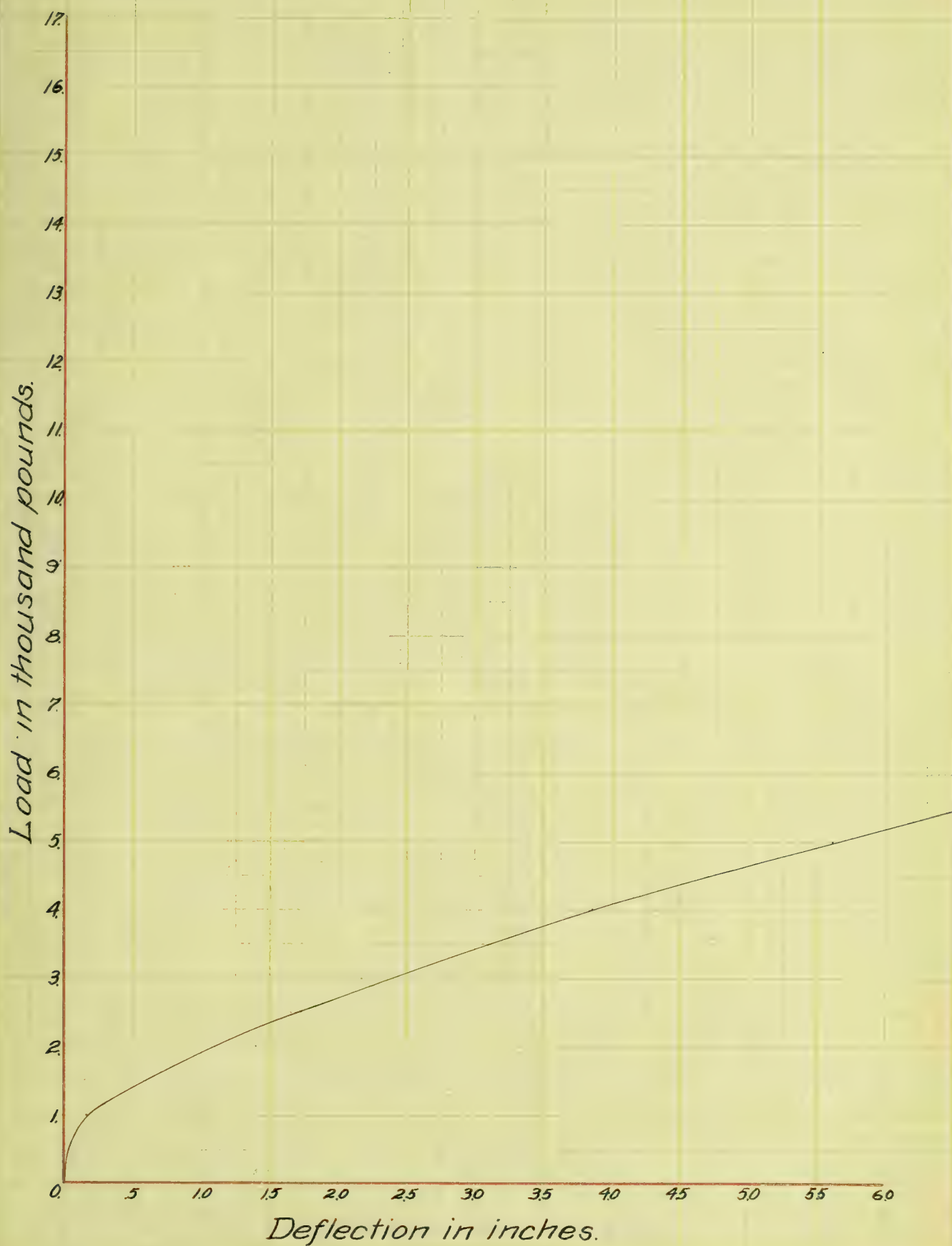
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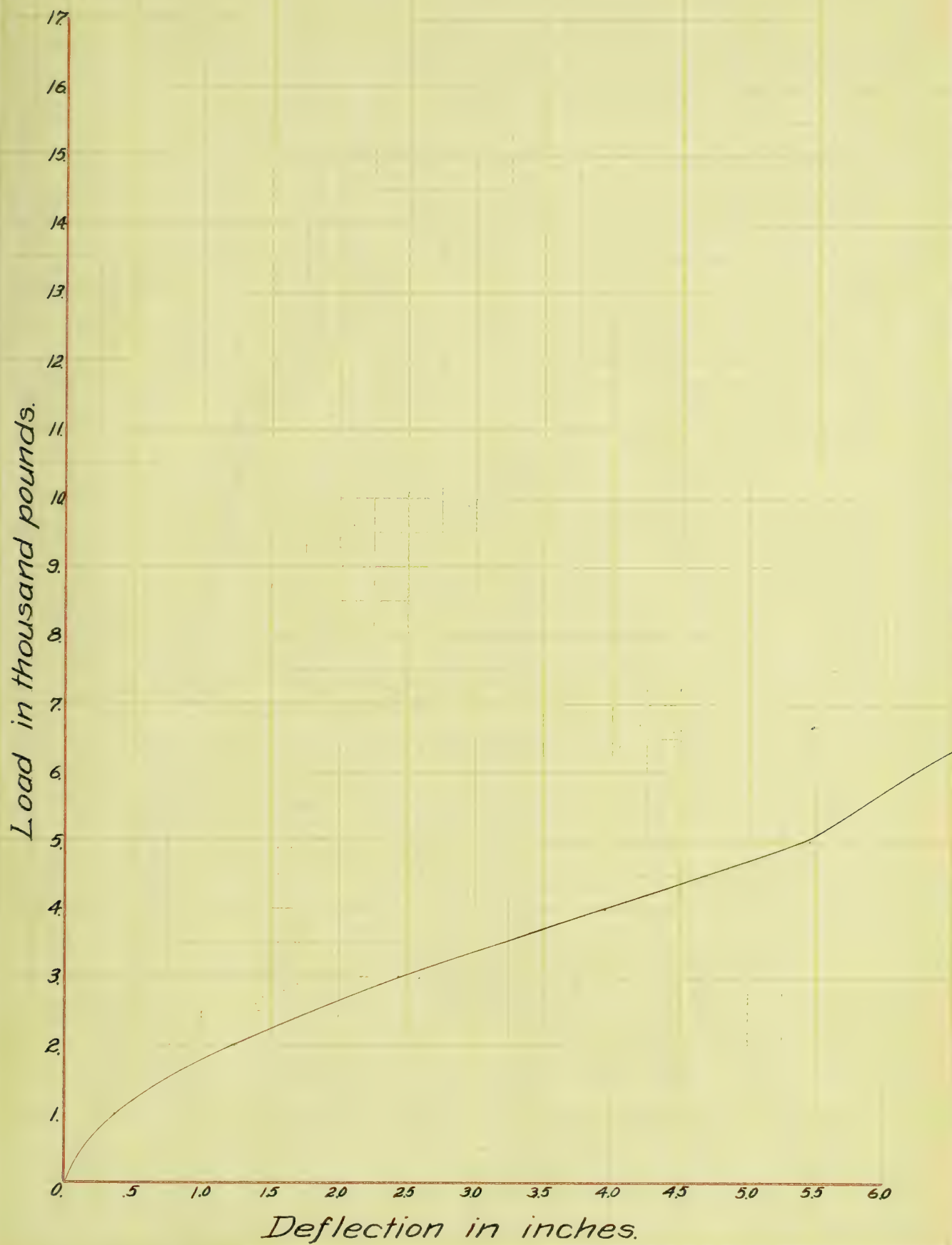
Beam #5a.

Beam #6.

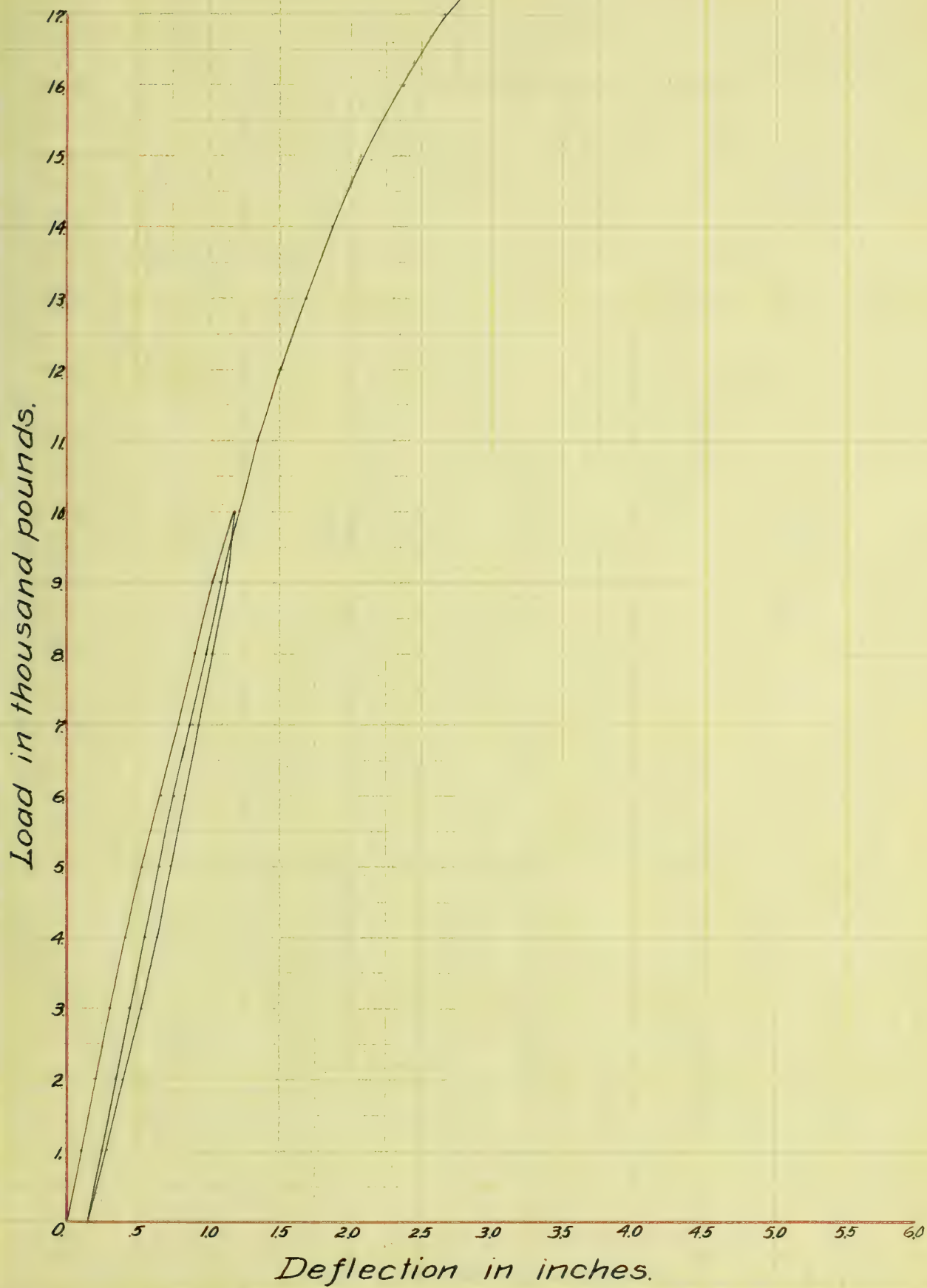
Beam #7.

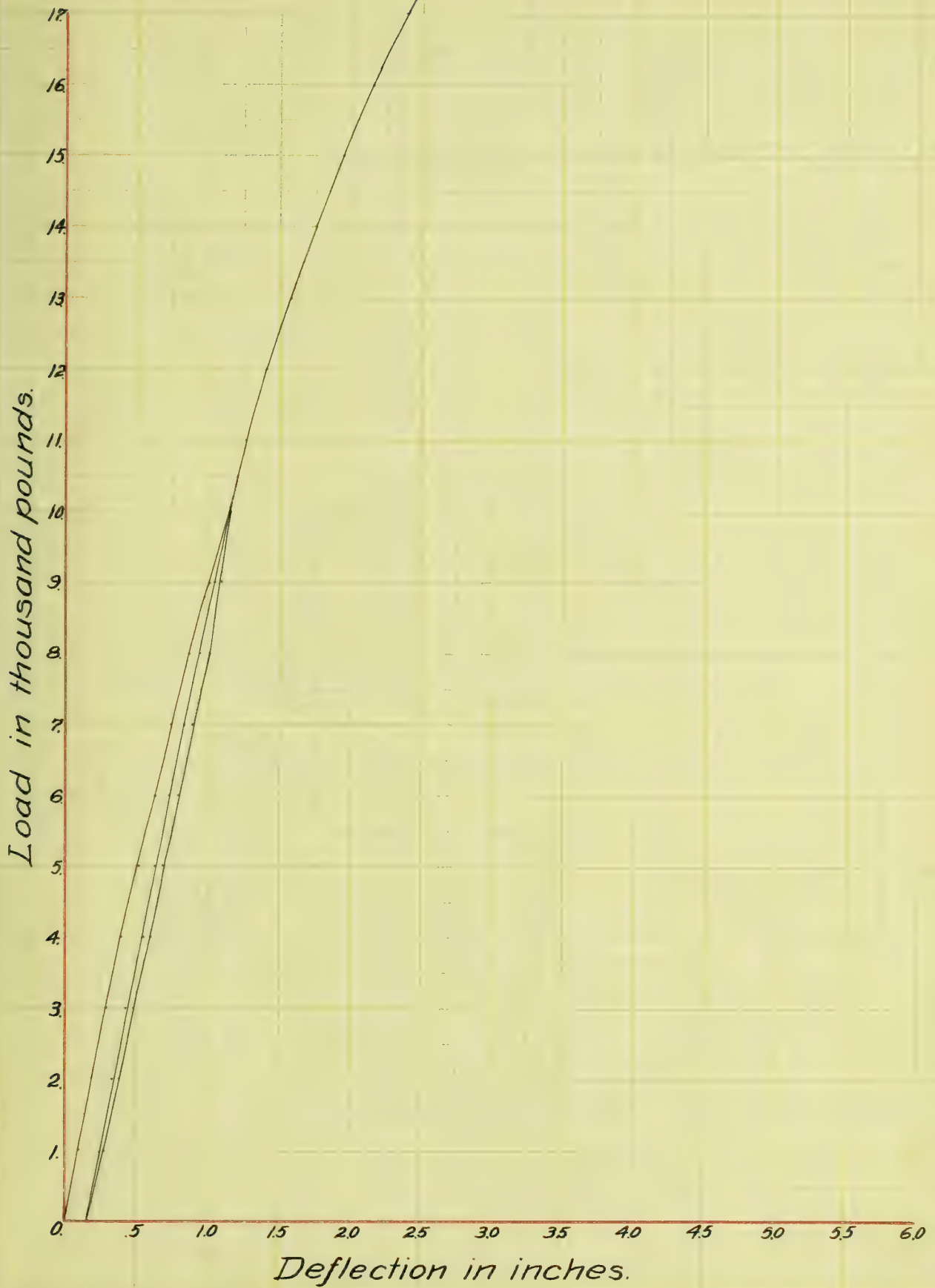
Beam #8.

Beam #9.

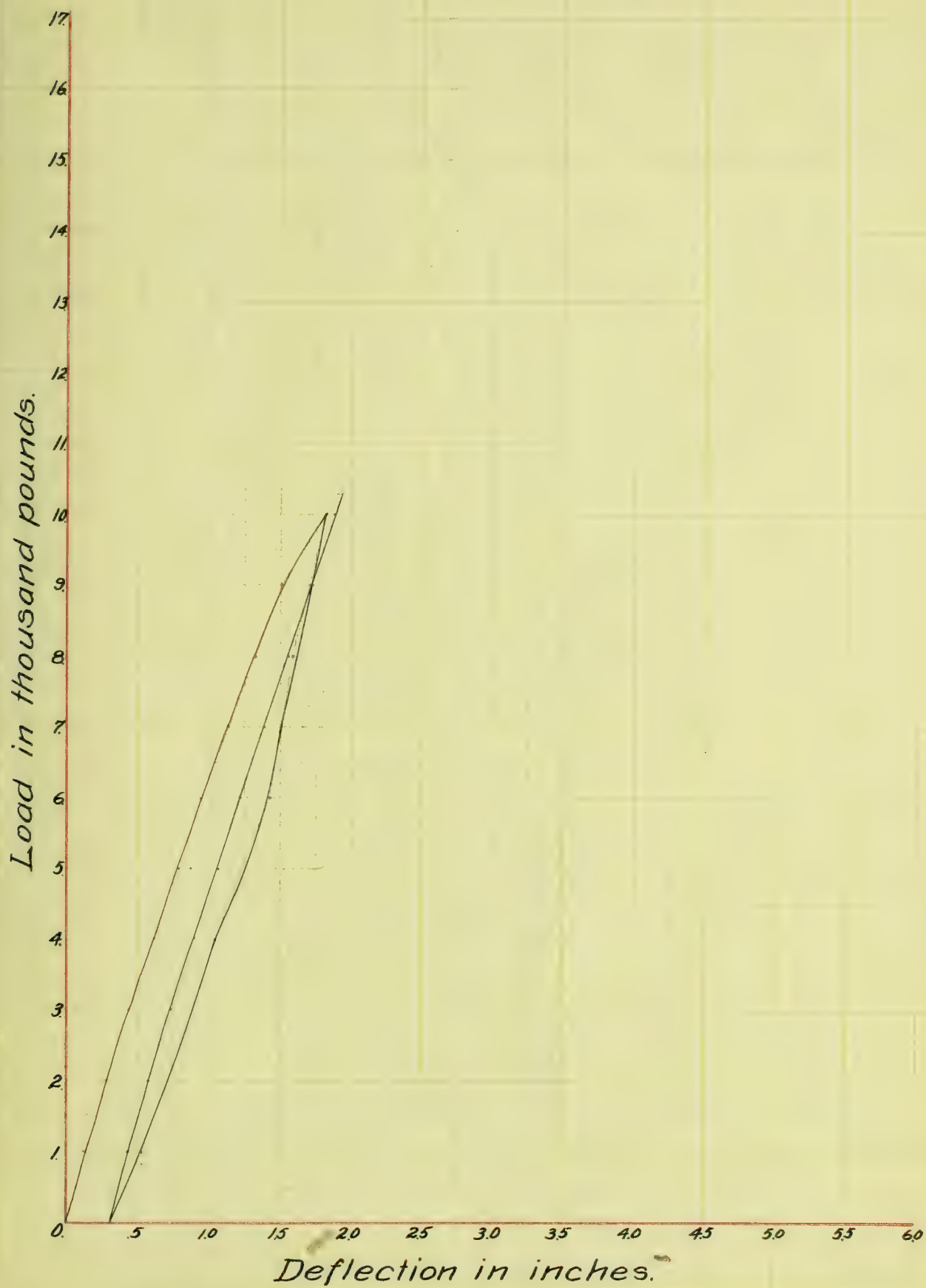
Beam #10.

Beam #11.

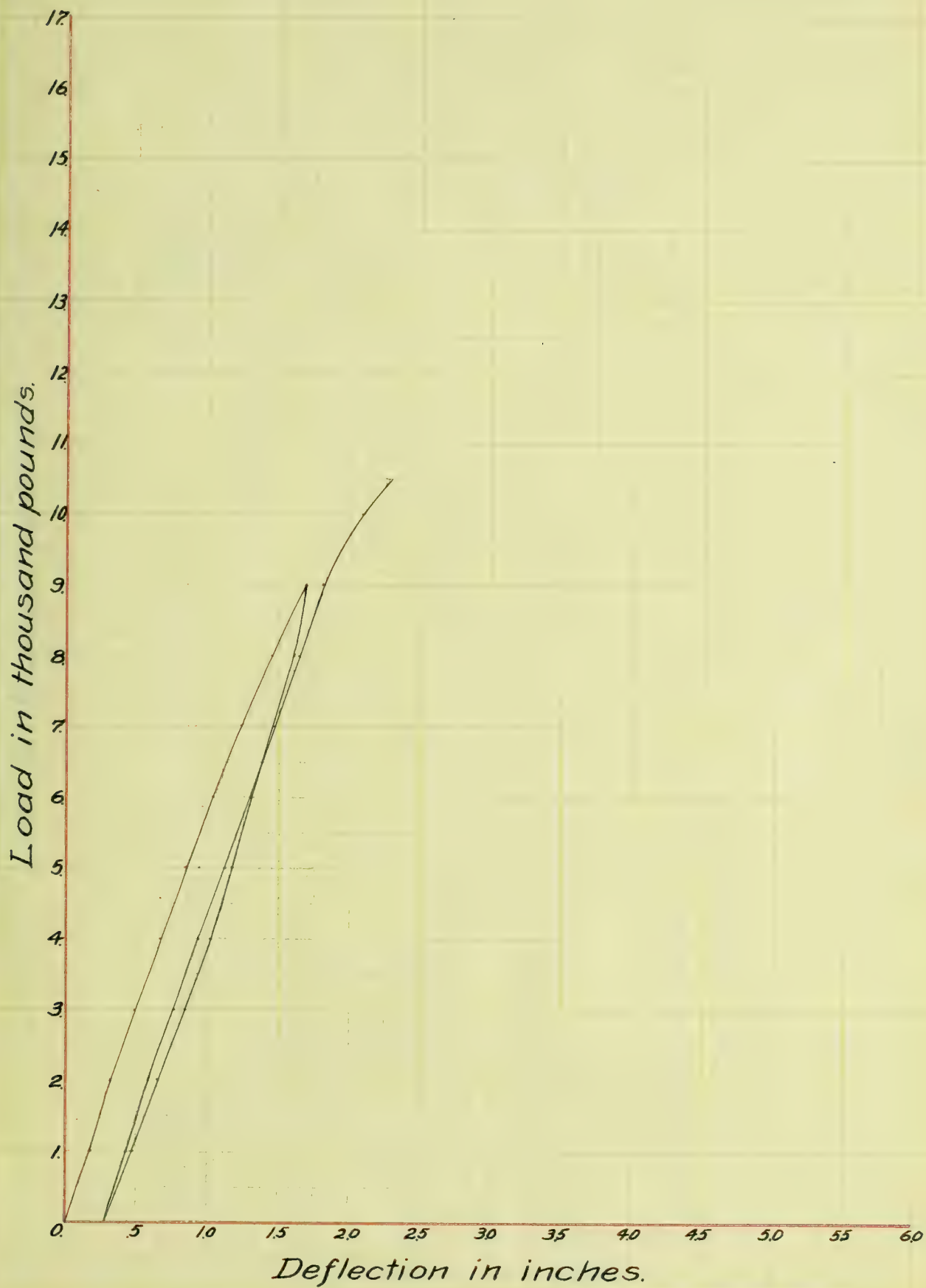


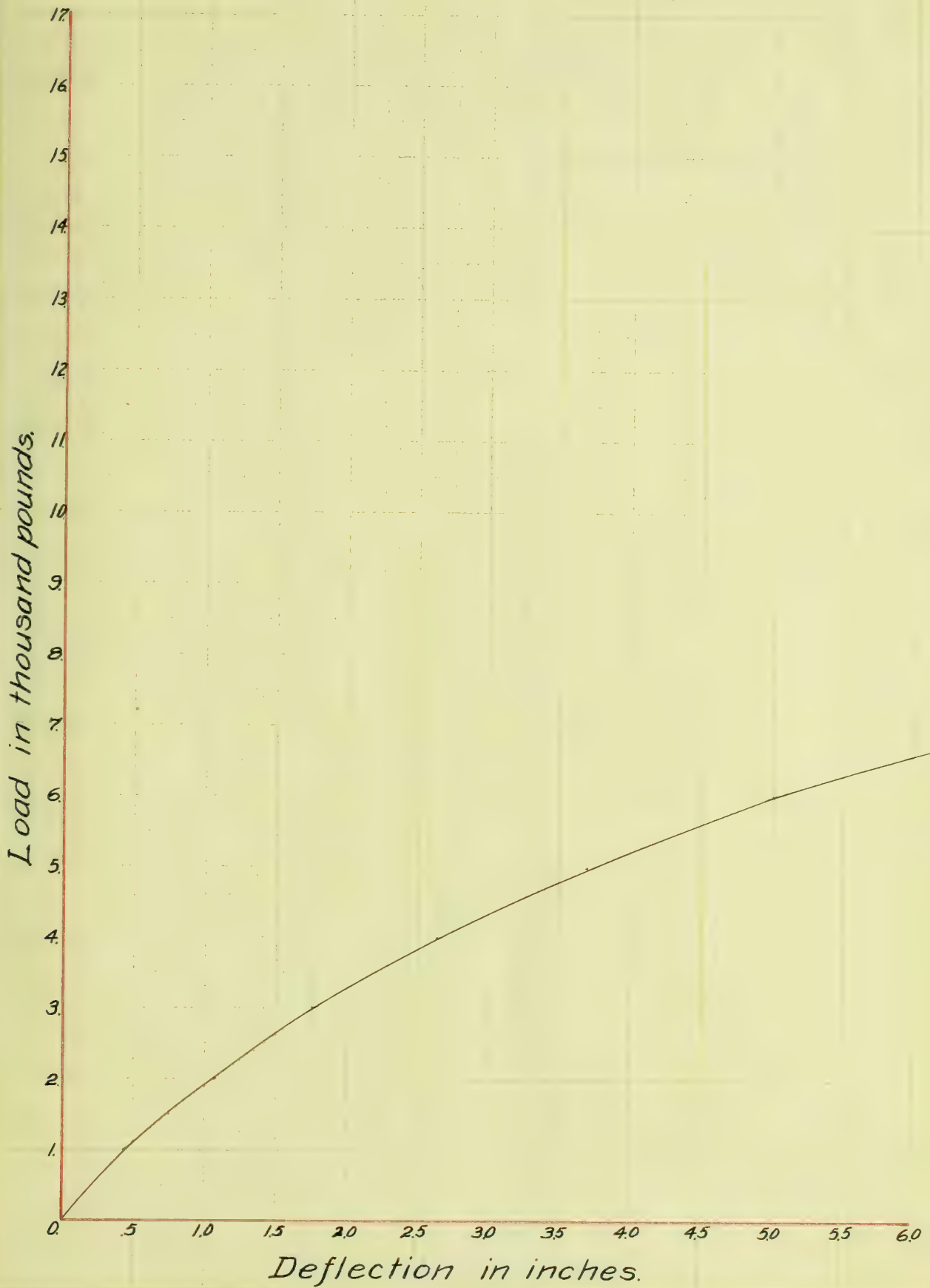
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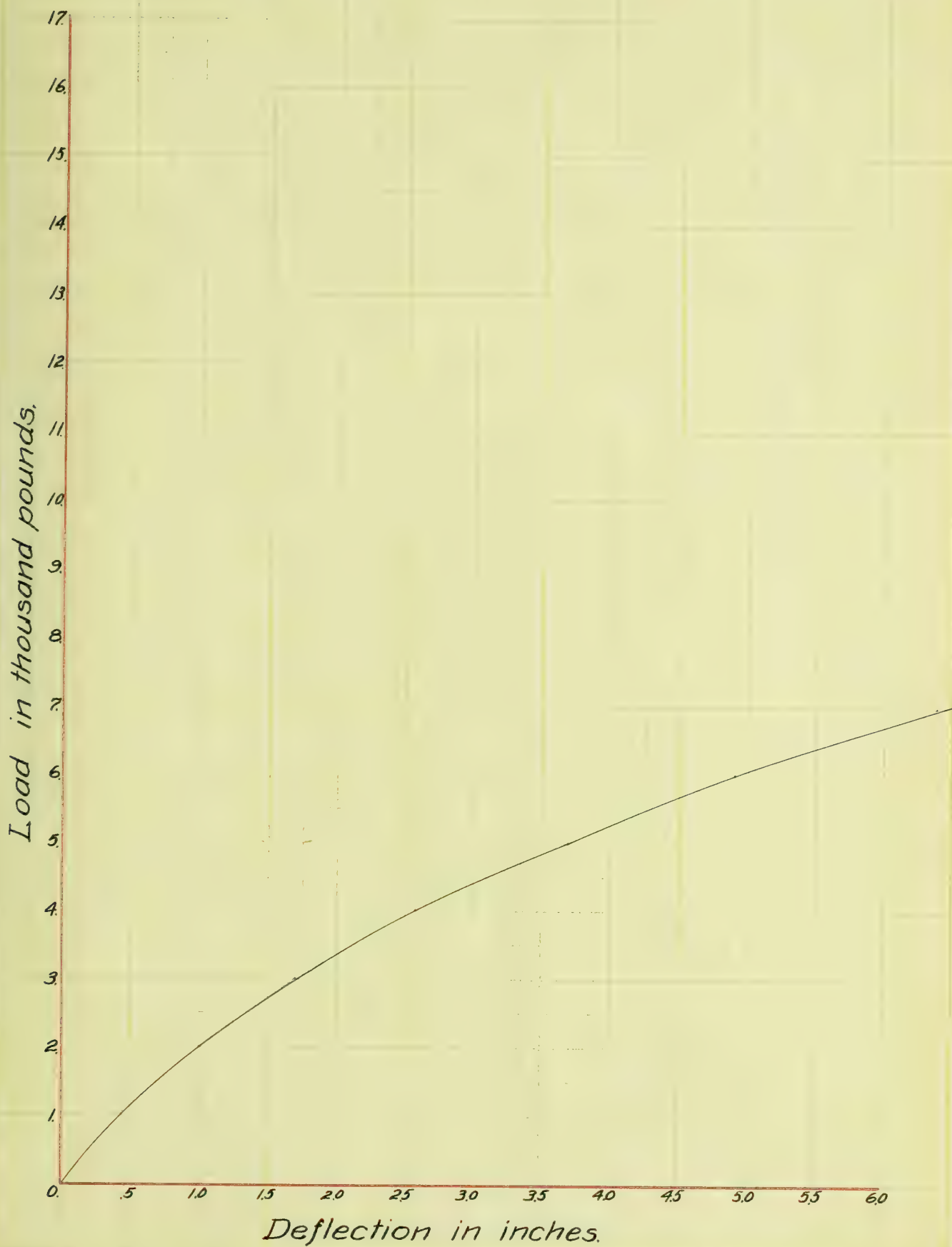
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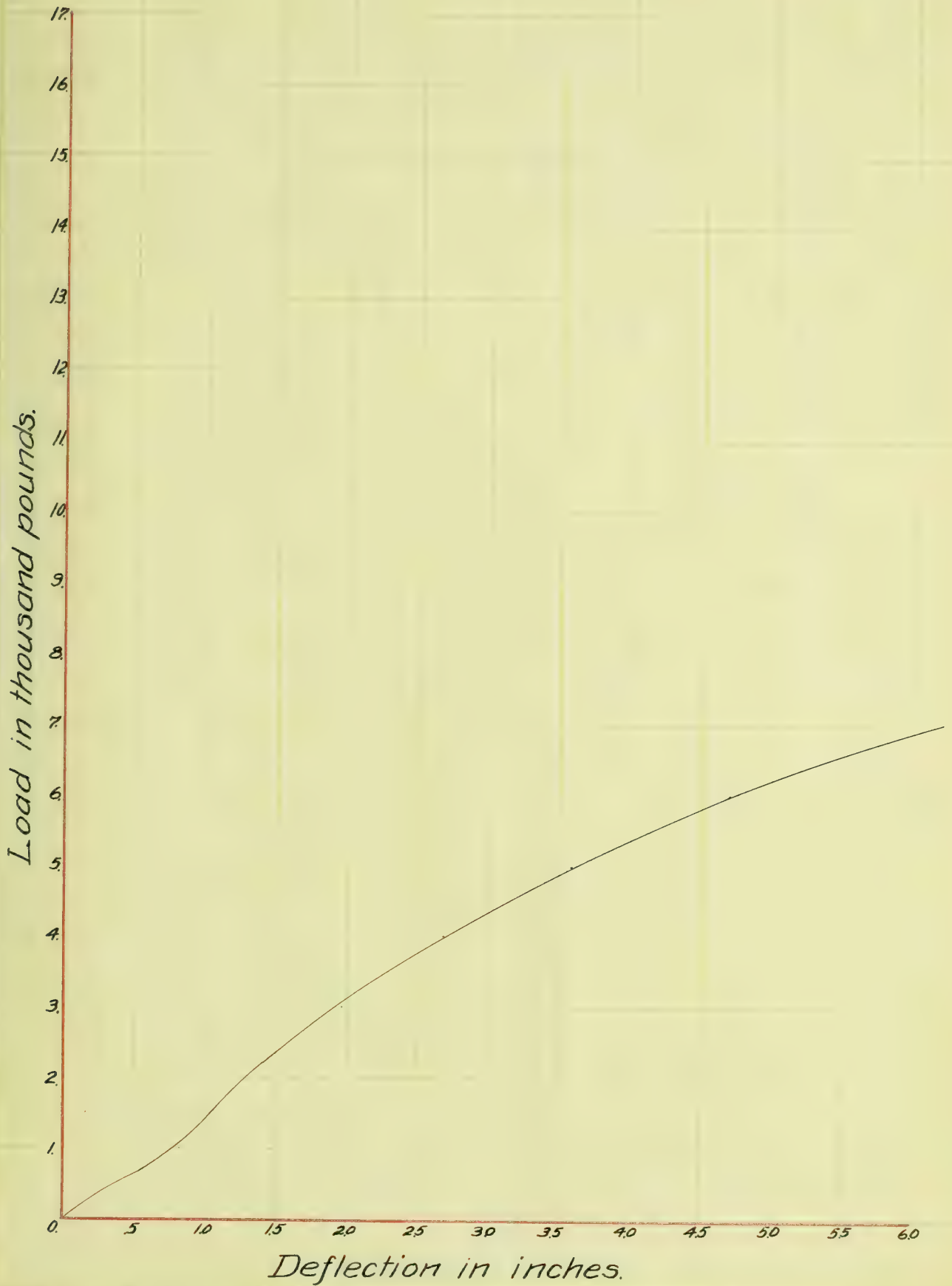


Beam #14.



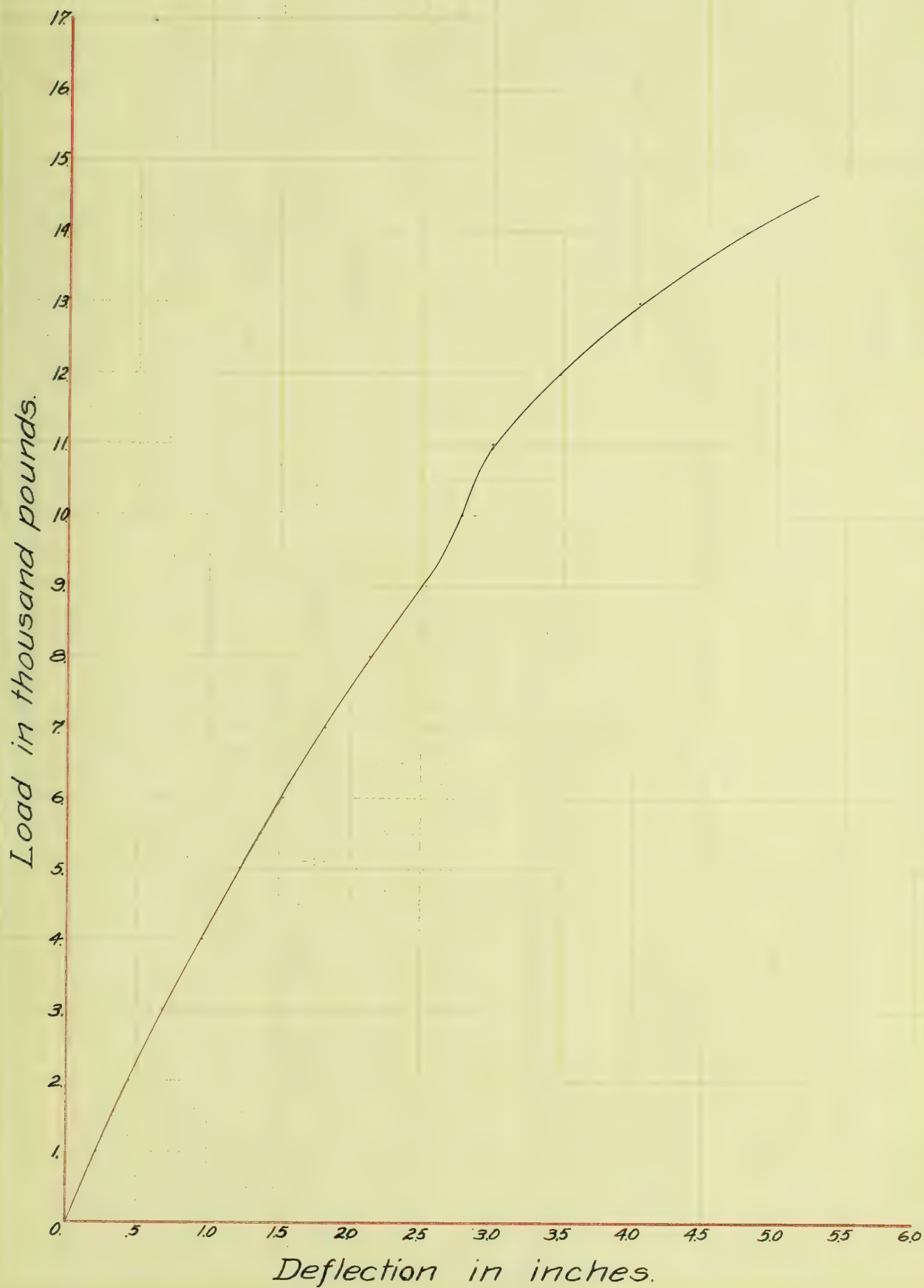
Beam #15.

Beam #16.

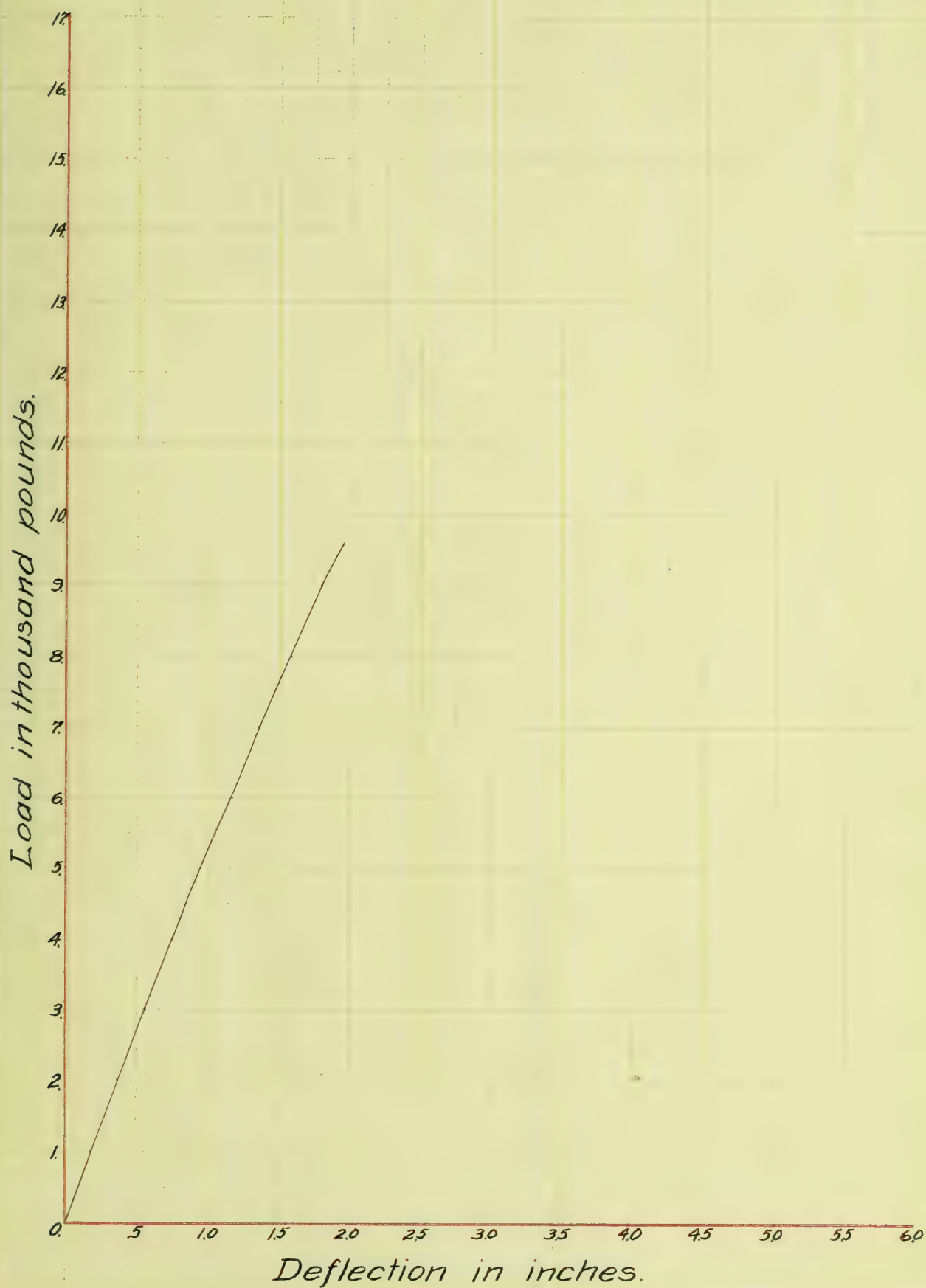
Beam #17

Beam #18.

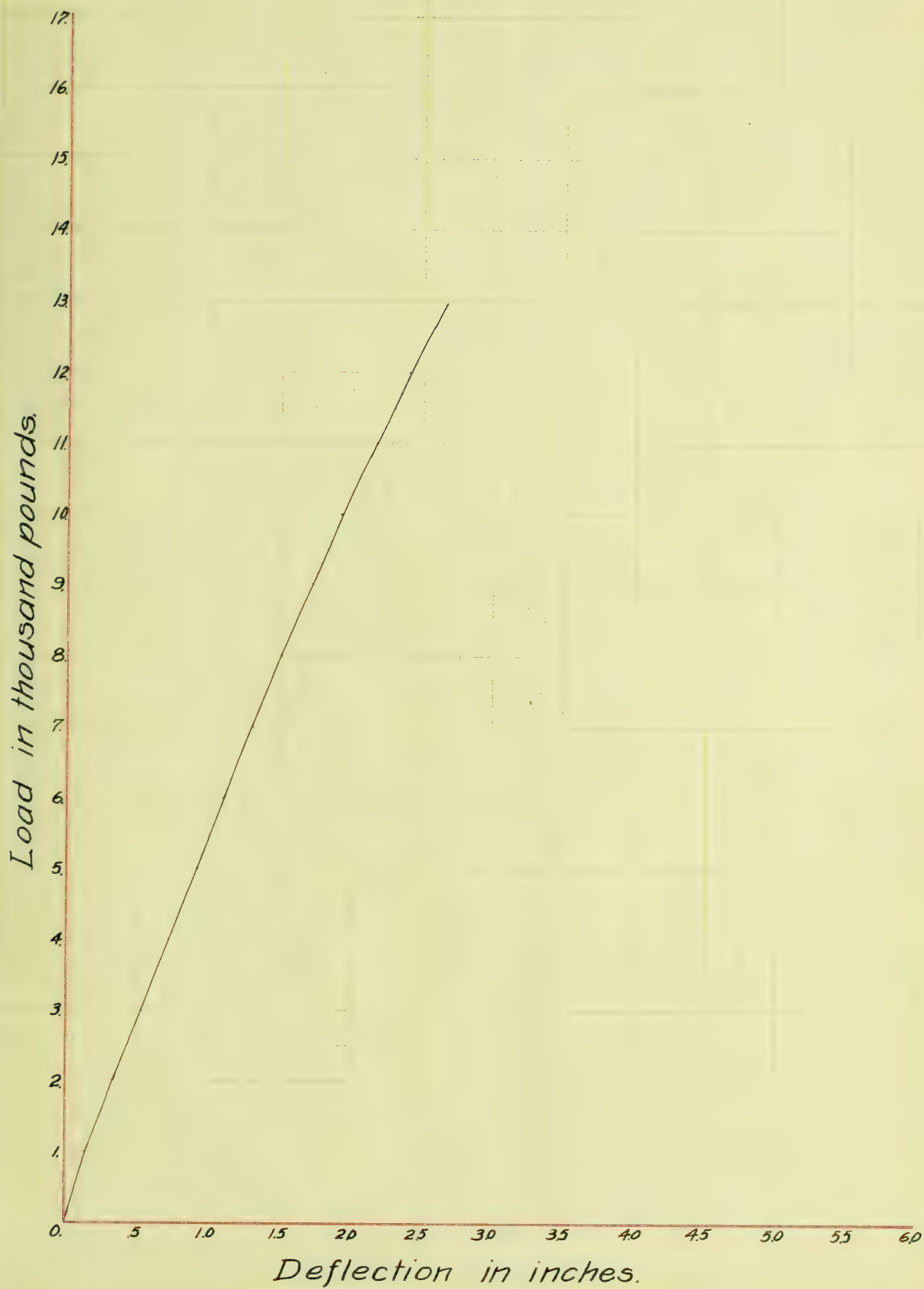
Beam # 19.

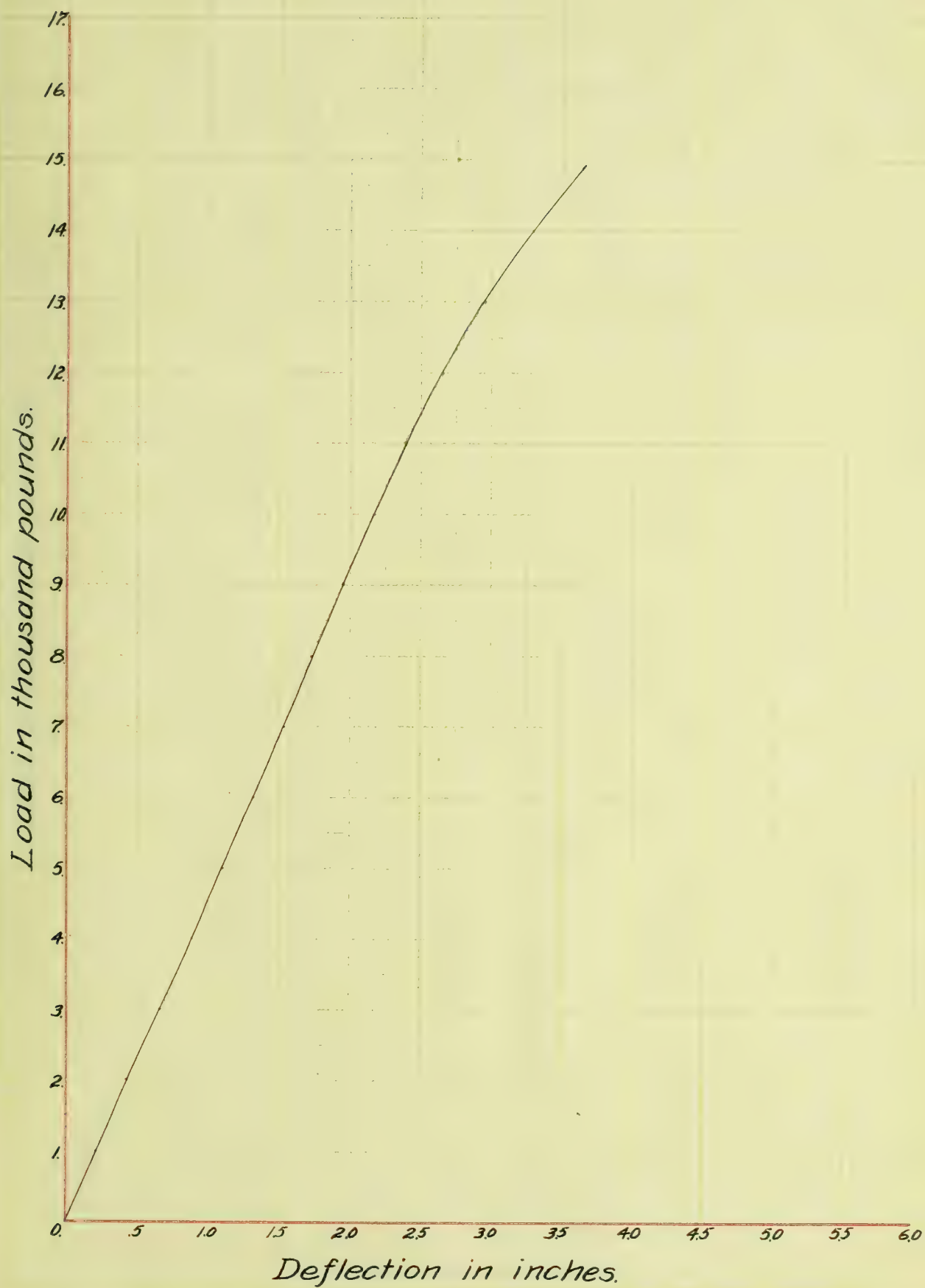
Beam #20.

Beam #21.

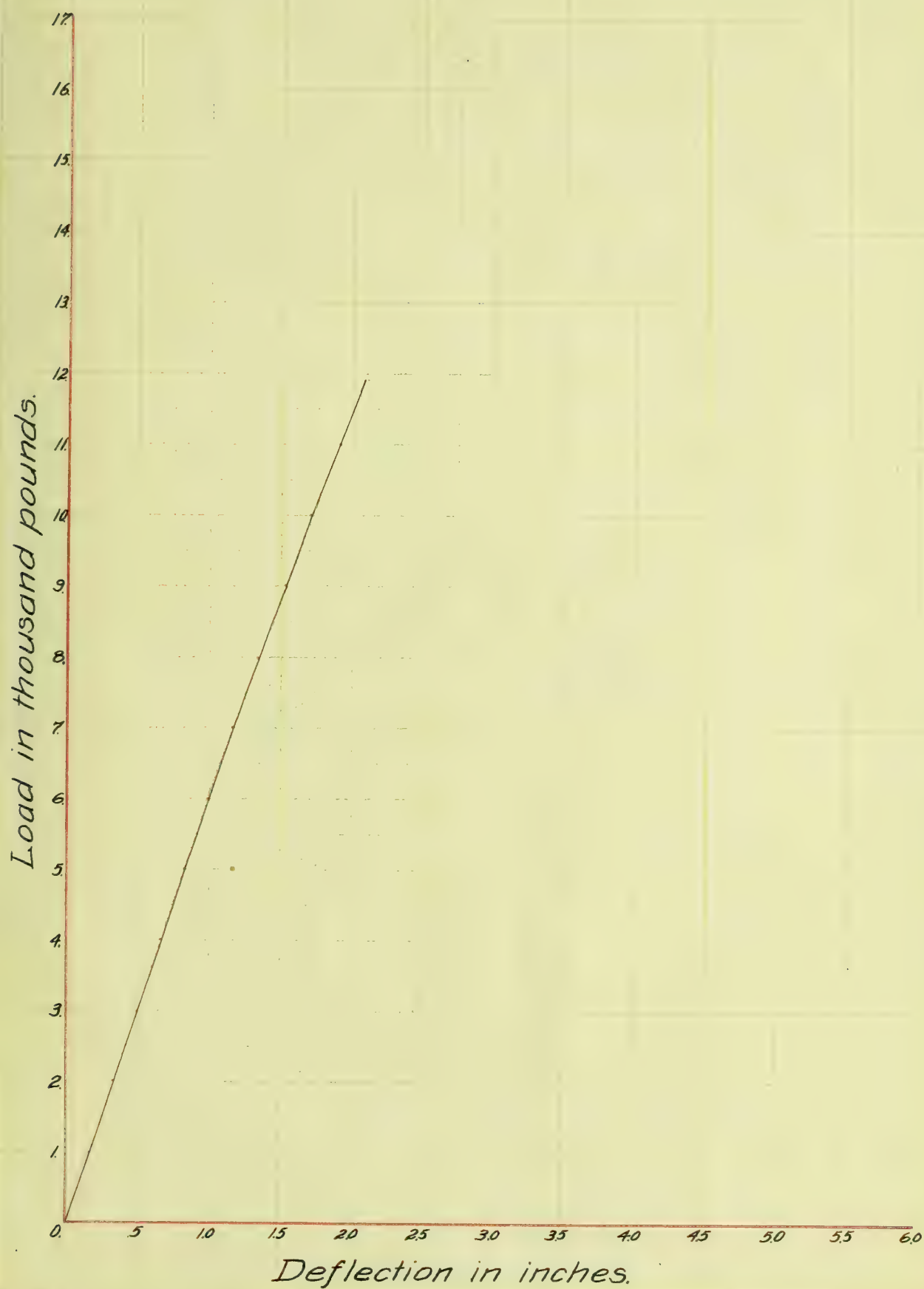


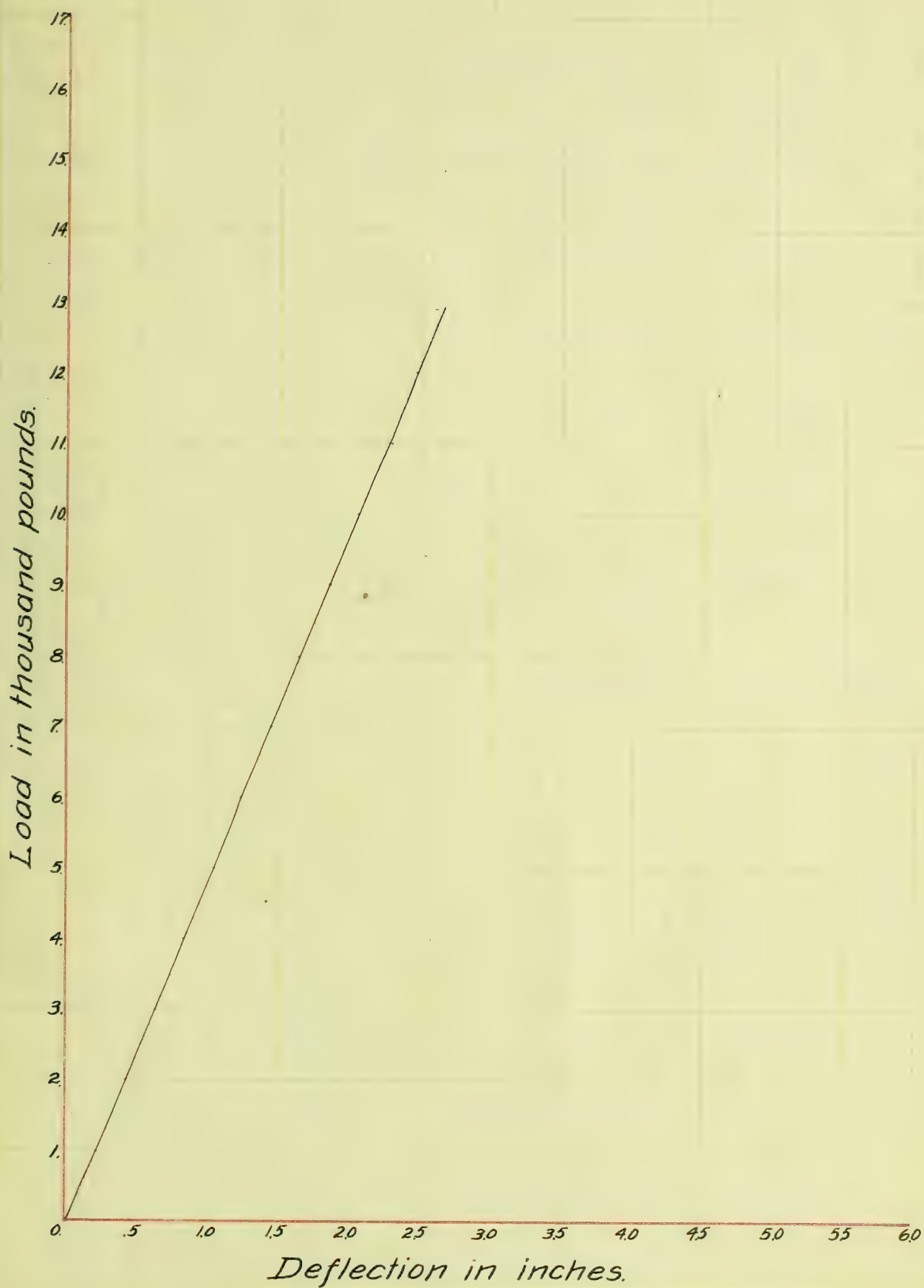
Beam #22.

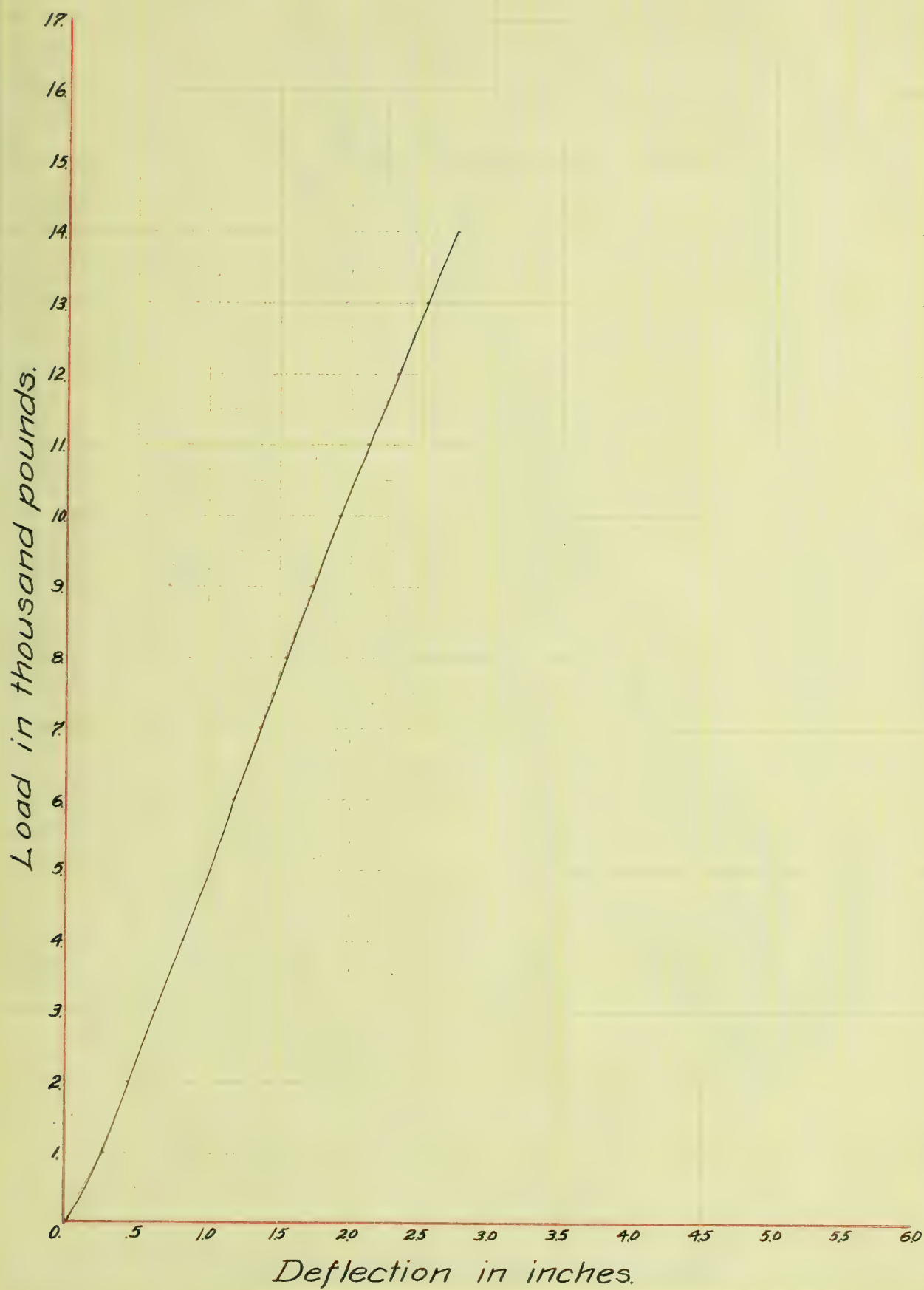


Beam #23.

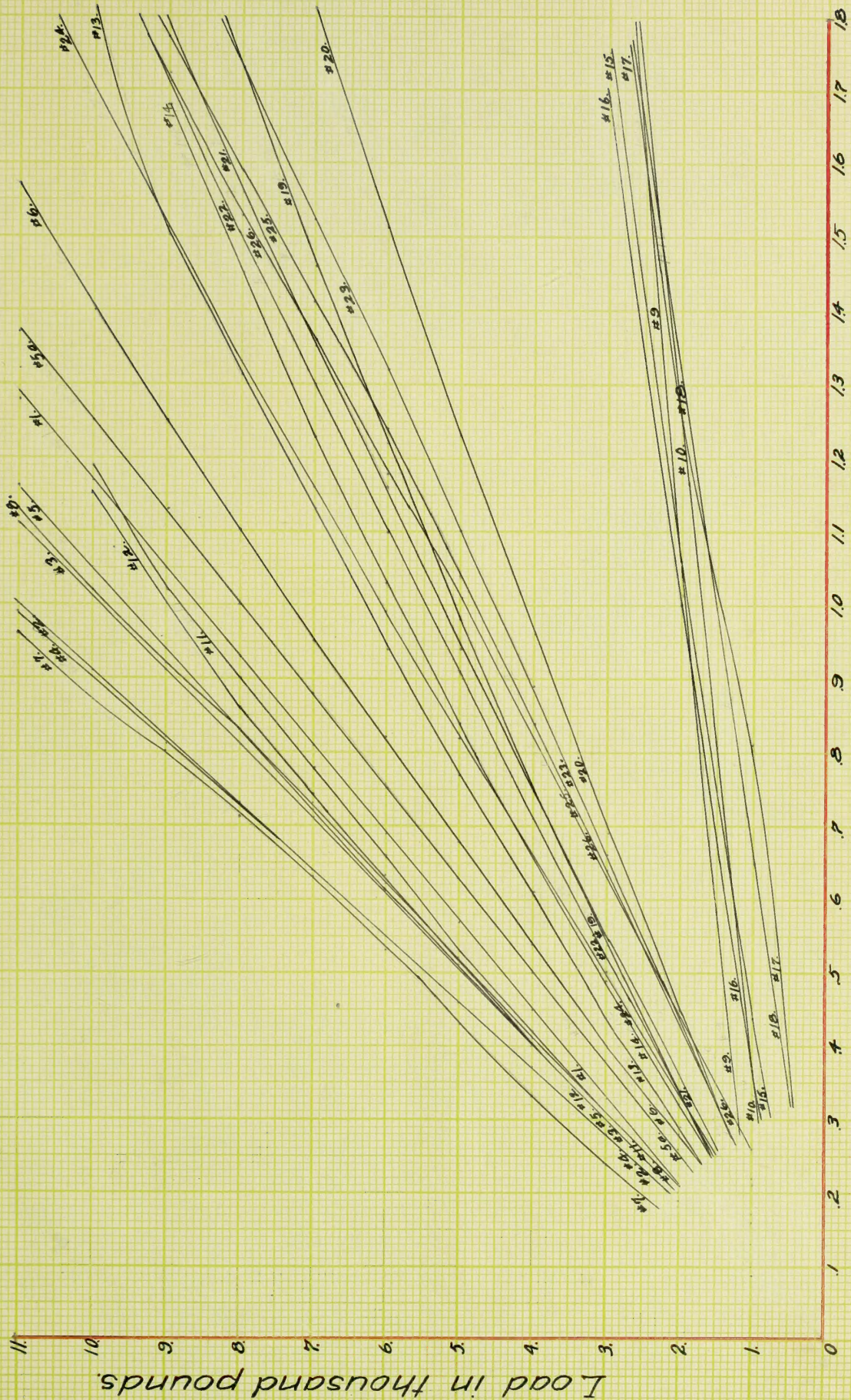
Beam #24.



Beam #25.

Beam #26.

Comparison of Beams.

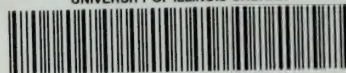


Deflection in inches.





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